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ACTING SECRETARY, R.E. INSTITUTE.

PAPER 7.

CONSTRUCTION OF BRIDGES
TO TAKE HEAVY MOTOR
TRANSPORT.

BY

CAPT G. C. GOWLLAND, R.E.

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CONSTRUCTION OF BRIDGES TO TAKE HEAVY MOTOR TRANSPORT.

THE loads that have to be carried by military bridges have increased enormously with the advent of heavy motor transport. Conditions in civilized countries often entail the bridging of large gaps, where no intermediate support is possible. The field engineer will have to replace demolished bridges with semi-permanent work capable of taking all military traffic. The material will be usually that found near the site, or, failing that, material which can be quickly brought up. This material will usually be wood, steel or iron bars of small dimensions, nails, coach screws, bolts, etc.

It is not practical to have standard pattern bridges as each replacement will have to be worked out on its own merits, the design, etc., will probably have to be subordinated to the material that can be most easily obtained. Everyone, then, who may have to tackle a bridging problem should have such a knowledge of the subject as to be able to suit his design to his material and not *vice versa* as is the case if standard pattern bridges are relied on. The heaviest self-propelled vehicle found in use on the road at the time of writing—August, 1915—is the Lion Steam Tractor. These would not be met with in convoys and would be passed over singly. The heaviest vehicle likely to be met in convoys is the Foden Steamer. The latter is taken as a basis for calculation. Its weight may be taken as 12 tons live load, of this 8 tons comes on the hind and 4 tons on the fore axle. The distance between axles is 14 ft.

In replacing a demolished bridge it is undoubtedly the best practice to make the replacement capable of taking any load that can be brought on it. The disadvantages of having a weak bridge that can only take one heavy vehicle at the time are manifold. Lorries move about in convoys, often at night. On a dark night confusion is caused in passing vehicles over a bridge one by one. Much time is lost if a whole convoy is held up in this way, and there is also the danger that more than one lorry may get on the bridge and so cause a failure. The only advantage gained by making a bridge capable of taking only one vehicle is, that a certain amount of material is saved. The saving is, however, surprisingly small and does not count for much. The conclusion therefore is that all

replacements should be made to take a train of the heaviest tractors. If this is done the bridge is open to all traffic with no restrictions except in the case of Lion Steam Traction Engines which must be passed over one by one. This is obviously most satisfactory for all concerned. The greatest load that can come on a bridge is a train of lorries as shown in *Fig. 1* and all bridges must be constructed to carry this train of loads. If the lorries are much closer than the train shown in *Fig. 1* they will be moving very slowly and so the live load factor will be reduced and the bridge will still be sound. This train can either be taken on plate girders made entirely of planks with bolts, nails, etc., for fastenings, or else an N girder, or some form of lattice girder may be used. These N, or lattice girders may be made entirely of wood or if obtainable, steel or iron tie-bars may be worked in.

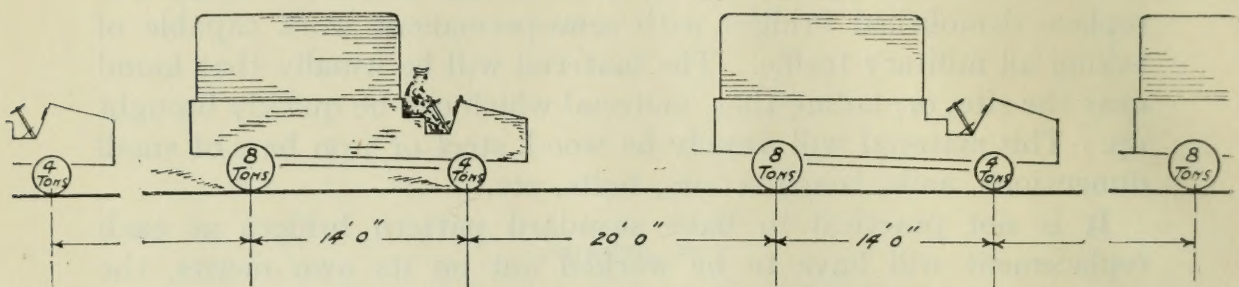


FIG. 1.—*Train of Lorries crowded on a Bridge.*

The booms will usually be of wood, rails can be used for this, but they take longer to put together.

The following are general rules for the dimensions of girder bridges:—The depth of girders should be at least $\frac{1}{14}$ th of span for stiffness. If the depth is over 7 ft. the girders become difficult to handle. For N or lattice girders a 5-ft. depth for spans from 30 ft. to 50 ft. will work out well a 6-ft. depth for 60-ft. span and a 7-ft. depth for spans from 70 ft. to 100 ft. Plate girders can conveniently be made shallower than this. The headroom above the roadway should be 14 ft. 6 in. The wheel track 8 ft. 6 in. between wheel guides. In bridging navigable waterways the headroom above the highest navigable water level is to be 12·13 ft. or 13·12 ft. above mean water level.

The method of carrying the roadway must now be decided. There are three ways of doing this:—

- (1). Roadway placed directly on the top boom.
- (2). Transoms put across the top boom at the panel points with roadbearers above transoms. This brings the loading directly on the panel points.
- (3). Transoms put across the bottom boom at the panel points with roadbearers as above.

Of these three methods the third must be discarded at once unless steel beams are available for transoms. The following argument will prove this :—

Suppose we decide to run the roadway on roadbearers supported on transoms placed on the bottom boom, using two girders. The distance between wheel guides must be 8 ft. 6 in. A lorry passing over the bridge may have one wheel hard up against the wheel guide. At least 9 in. must be allowed between the wheel guide and the girder. Therefore a lorry in the most unfavourable condition in regard to the transoms will give a loading as in *Fig. 2*.

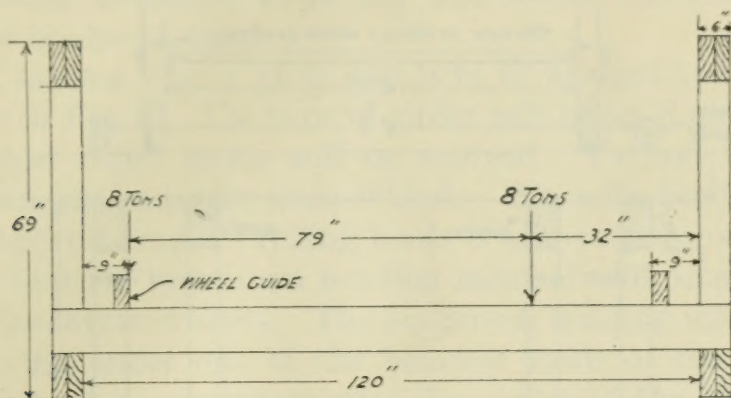


FIG. 2.

Using a factor of 2 for live load on the transom the bending moment in *Fig. 2* is 462,873 inch-lbs.

Suppose only 9-in. $\times$ 3-in. timber is available and taking $r=1200$, then

$$462873 = \frac{1}{8} \times 1200 \times b \times d^2.$$

d cannot be greater than 9 in. since 9-in. $\times$ 3-in. is the largest scantling.

$$\therefore \frac{462873 \times 6}{1200 \times 9 \times 9} = b.$$

Whence $b = 28.5$ ".

Therefore the transoms would have to be built up of nine 9-in. $\times$ 3-in. On these transoms would come heavy roadbearers and above this the roadway. This would make the superstructure unduly heavy besides taking much time and material.

In Case (2) the transoms will not undergo such a large bending stress as in Case (3). It has been proved by experience that lorries may be safely rolled directly on the top boom of girders calculated from panel loadings and therefore the addition of transoms and roadbearers on the top boom appears unnecessary and only leads to extra weight in the superstructure.

The conclusion therefore is that except under exceptional circumstances it is best to carry the roadway on the top boom direct. In adopting this method the local bending moment due to a heavy axle being between two panel points must not be forgotten. This question will be gone into later. It is often much easier to construct a bridge made up of four girders instead of two. This is much easier to arrange if the roadway is run on the top boom. Two girders can be placed under each wheel guide so that with a lorry placed centrally between the wheel guides one wheel will come between each pair of girders (see *Fig. 3*).

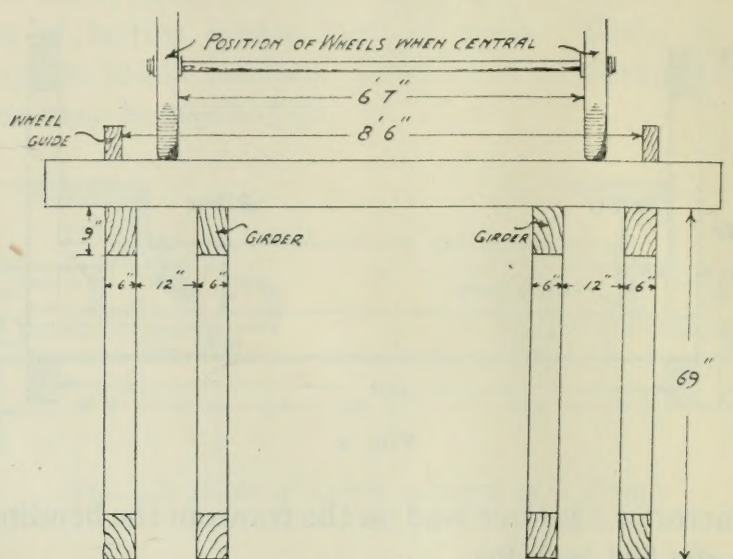


FIG. 3.—Roadway on Top Boom with no Transoms. Four Girders used.

If all the girders are well braced together it is safe to conclude that each girder does its fair share of work. By comparing *Fig. 3* and *Fig. 4* the advantage of running the roadway on the top boom is at once apparent. In *Fig. 4* the two outside girders can never take their full load.

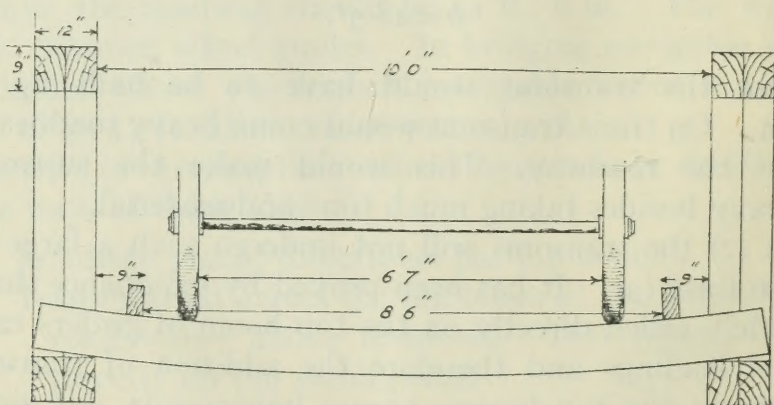


FIG. 4.—Roadway on Bottom Boom. Position of Wheels when central.

Considerations of headroom below the bridge may sometimes make it impossible to run the roadway on the top boom, in which case another method must be adopted, steel or trussed transoms being used.

The ruling dimensions for girders have already been given. The best method of carrying the load and the greatest load to be carried has also been decided. The method of illustrating the design of different forms of girders will be to take various concrete examples and work out the solutions in detail. It is necessary to make some assumptions as to the material that will be available for the construction of the girders. It is assumed that 9-in. $\times$ 3-in. fir is the largest timber available while flat and round bar steel of small dimensions is also at hand.

We will assume that a 50-ft. gap is to be bridged to take a train of loads as in *Fig. 1*. The type of girder will be an N girder. From this N girder other forms will be evolved. To find the greatest bending moment under the train of loads. Since the bending moment diagram under a train of rolling loads is always polygonal in form, it is clear that the maximum bending moment will occur under one of these concentrated loads. The maximum bending moment nearly always occurs under one of the heaviest loads of the train when this heavy load is somewhere near the centre of the bridge and as many loads as possible are on the bridge. In this case it is fairly obvious that the greatest bending moment will occur under the load A (*Fig. 5*) when that load is somewhere near the centre.

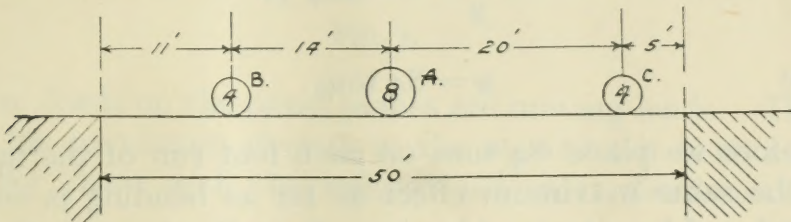


FIG. 5.

The actual position of the loads when they produce the greatest bending moment is found by calculating the centre of gravity of all the loads on the bridge and then arranging the loads so that the centre of the bridge bisects the distance between the load under which the M_{ff} occurs and the centre of gravity of all the loads.

In this case let x be the distance of the centre of gravity of the loads B, A, C, from the load B.

$$\text{Then} \quad x \times 16 = 8 \times 14 + 4 \times 34.$$

$$\text{Whence} \quad x = 15.5'.$$

Following the rule given, the position of the loads when producing the maximum bending moment is as in *Fig. 6*.

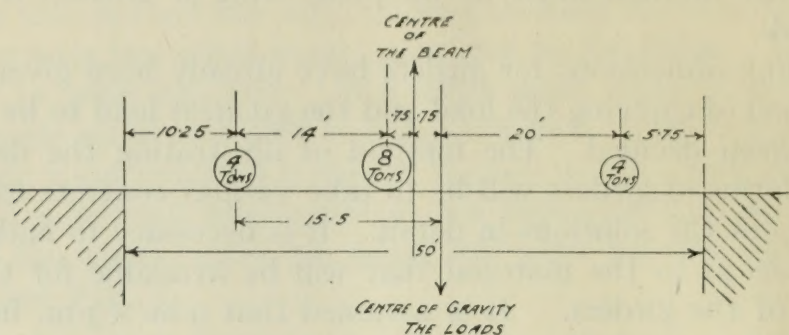


FIG. 6.

This position can also be obtained from the formula given on p. 24, *Military Engineering*, Part III.

The bending moment under the loading in *Fig. 6* is 132.18 foot-tons and is the greatest that can be produced by the train of lorries. This bending moment is caused by a live load and must be reduced to equivalent dead load. To reduce to equivalent dead load multiply by either 2 or $\frac{3}{2}$. The merits of these two factors will be discussed later. In this case use the factor 2.

Then the M_F due to equivalent dead load is 264.36 foot-tons.

Let w tons per foot-run be placed on the bridge, the weight w being such that it produces an M_F in the bridge of 264.36 foot-tons.

$$\text{Then} \quad \frac{w \times 50 \times 50}{8} = 264.36$$

$$\text{Whence} \quad w = .84 \text{ tons.}$$

If therefore we place .84 tons on each foot run of the bridge we produce the same maximum effect as far as bending is concerned that is produced by a train of lorries. By collecting this distributed load at the panel points, for the purposes of calculation, the result will not be much altered.

The distributed loads that will produce the maximum bending moment for all spans from 10 to 100 ft. are shown in the curve shown on *Plate I*.

From this curve the panel-point loading for any bridge can be quickly arrived at, but this panel-point loading will only give the maximum bending moment and not necessarily the maximum shear. The question of shear is dealt with later. The curve shows equivalent dead loads in tons per foot-run, the live load being multiplied by 2 to reduce to equivalent dead load. Once the panel-point loading is found the stresses in all members can be worked out by the method of sections.

In the case under consideration the span is 50 ft. Therefore convenient panels will be 5 ft. $\times$ 5 ft. Suppose it is decided to use four girders, then panel-point loading for each girder is :—

$$\frac{\text{Length of panel in feet} \times \text{load per foot-run for 50-ft. span (from curve)}}{\text{Number of girders.}}$$

In this case panel-point loading is $\frac{5 \times 84}{4}$ tons, say 1 ton per panel point on each girder. This is due to equivalent dead load. The actual dead weight of the girder must be taken into consideration. A bridge of 50-ft. span made up of four girders complete with sleeper roadway, wind ties, handrail, cross-bracing, etc., weighs about 10 tons. This figure is taken from an actual bridge. The weight of the girders only for 50-ft. span varies from 20—30 cwts. per girder according to the type employed. The sleeper roadway weighs about $3\frac{1}{2}$ tons while cross-bracing, handrail, etc., brings the weight up to 10 tons. Therefore with four girders each one takes $2\frac{1}{2}$ tons dead load.

Each girder is loaded as in *Fig. 7*. The dead load is assumed to act at the top of the panel points.

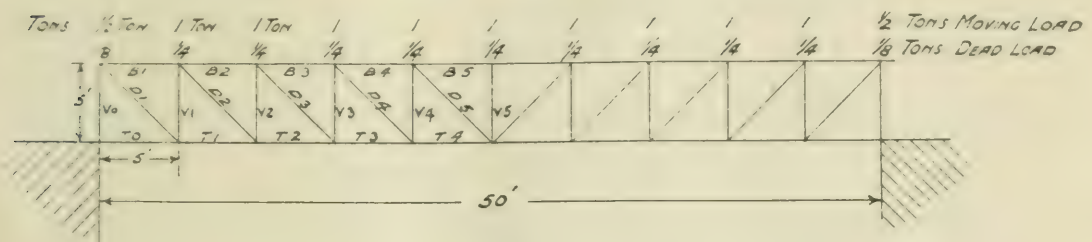


FIG. 7.

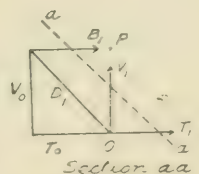
The 1-ton loads on the panel points are moving loads. Therefore to obtain the maximum stresses in the members it is necessary to calculate for them with the rolling load most disadvantageously placed.

The stresses in $B_1, B_2 \dots B_{10}$ and in $T_0, T_1 \dots T_9$ are maximum with the girder fully loaded. The verticals and diagonals are not always under maximum stress when the girder is fully loaded.

The stress in $T_0=0$.

To find the stress in B_1 and T_1 take a section aa .

Take moments about O. Clockwise moments being negative, counter-clockwise moments being positive. The compression sign being $-$. The tension sign being $+$.



Then $-B_1 \times 5 - (1 + .25)(1_0^1 + 1_0^2 + 1_0^3 \dots 1_0^9)5 = 0$.

$$\therefore B_1 = -5.625 \text{ tons or } -12600 \text{ lbs.}$$

Similarly by taking moments about P, $T_1 = +12600$ lbs.

To find B_2 and T_2 take a section bb .

Taking moments about O.

$$-B_2 \times 5 - (1 + .25) (1_0^1 + 1_0^2 \dots 1_0^8) - (1 + .25) (1_0^9 \times 10 - 5) = 0.$$

$$\therefore B_2 = -10 \text{ tons} = -22400 \text{ lbs.}$$

Taking moments about P.

$$T_2 = +10 \text{ tons} = +22400 \text{ lbs.}$$

To find B_3 and T_3 take a section cc .

Taking moments about O.

$$-B_3 \times 5 - (1 + .25) (1_0^1 + 1_0^2 + 1_0^3 \dots 1_0^7) - (1 + .25) (1_0^8 \times 15 - 5) - (1 + .25) (1_0^9 \times 15 - 10) = 0.$$

$$\text{Whence } B_3 = -13.1 \text{ tons} = -29400 \text{ lbs.}$$

Taking moments about P.

$$T_3 = +13.1 \text{ tons} = +29400 \text{ lbs.}$$

Similarly

$$B_4 = -15 \text{ tons} = -33600 \text{ lbs.}$$

$$T_4 = +15 \text{ tons} = +33600 \text{ lbs.}$$

$$B_5 = -15.62 \text{ tons} = -35000 \text{ lbs.}$$

To find $V_0, V_1, V_2, V_3, V_4, V_5$ and D_1, D_2, D_3, D_4, D_5 .

V_0 = the reaction at the abutment and is maximum under full load.

$$\therefore V_0 = -6.25 \text{ tons} = -14000 \text{ lbs.}$$

In section aa adding all the vertical forces the sum must be zero.

$$\therefore +V_1 + (1 + .25) (1_0^1 + 1_0^2 + 1_0^3 \dots 1_0^9) = 0.$$

$$\therefore V_1 = -5.625 \text{ tons} = -12600 \text{ lbs.}$$

and is maximum under full load.

$$\text{Also } -D_1 \sin 45 = V_1.$$

$$\therefore D_1 = +18000 \text{ lbs.}$$

and is maximum under full load.

Adding the vertical forces in section bb .

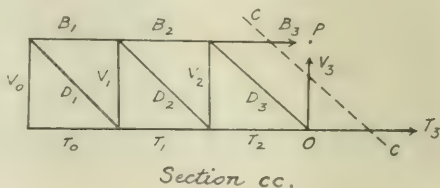
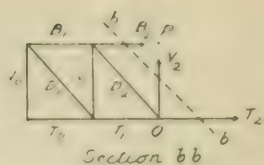
$$+V_2 + .25 (1_0^1 + 1_0^2 \dots 1_0^8 - (1 - 1_0^9)) + 1 (1_0^1 + 1_0^2 \dots 1_0^8) - 1 (1 - 1_0^9) = 0.$$

$$\therefore V_2 = \underbrace{- .875}_{\text{Due to dead load.}} \underbrace{- 3.6}_{\text{Due to moving load.}} \underbrace{+ .1}_{\text{Due to moving load.}}$$

Leaving out first the positive sign due to the moving load and then the negative sign due to the moving load

$$V_2 = -4.475 \text{ maximum.}$$

$$V_2 = -.775 \text{ minimum.}$$



That is the stress in V_2 varies from 4.475 tons compression to .775 tons compression and is maximum when the moving load extends over the bridge from the right abutment up to, and including the panel point where V_2 meets the top boom. V_2 is minimum when the moving load covers the bridge from the left abutment up to and including the panel point to the left of the junction of V_2 and the top boom.

$$-D_2 \sin 45 = V_2.$$

$$\therefore D_2 = +14336 \text{ lbs. max.}$$

V_2 is always in compression. D_2 is always in tension.

Adding the vertical forces in section cc .

$$+V_3 + .25 (1_0^1 + 1_0^2 \dots 1_0^7 - (1 - 1_0^8) - (1 - 1_0^9)) + 1 (1_0^1 + 1_0^2 \dots 1_0^7) - 1 (1 - 1_0^8) - (1 - 1_0^9) = 0.$$

Whence $V_3 = -3.425$ tons compression maximum or $-.325$ tons compression minimum.

Also $D_3 = +10916$ lbs. tension maximum or 1010 lbs. minimum.

V_3 is always in compression. D_3 is always in tension.

Similarly $V_4 = -2.475$ tons compression maximum but varies to $+.225$ tons minimum.

That is V_4 under certain loading is subject to tension.

D_4 varies from $+7930$ lbs. tension to -722 lbs. compression.

V_5 varies from -3570 lbs. compression to $+1960$ lbs. tension.

D_5 varies from $+5200$ lbs. tension to -2820 lbs. compression.

The stresses in the girder can now be written down and are shown in *Fig. 8*.

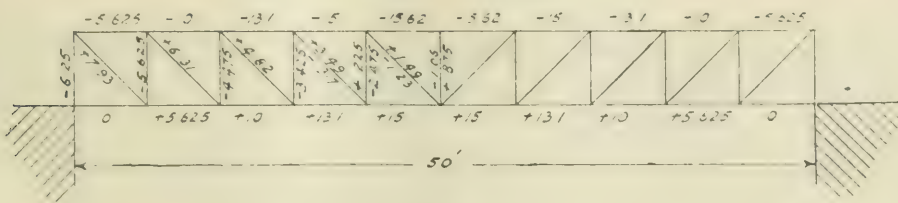
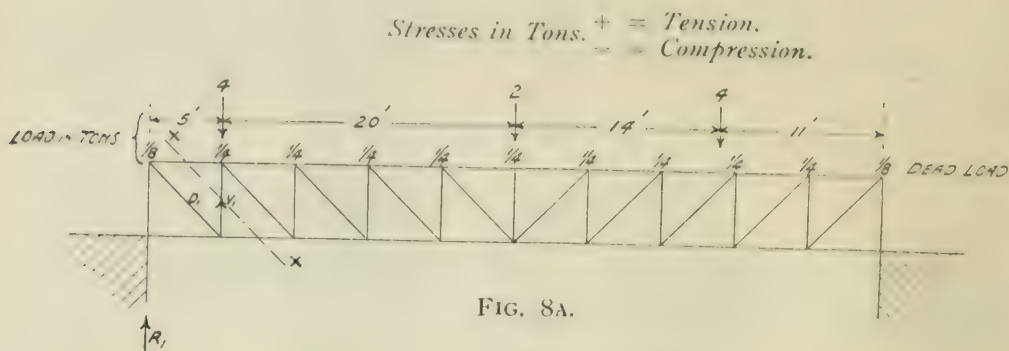


FIG. 8.

Plate II. shows the stresses in the girder worked out graphically. This only shows the maximum stress under full load and does not give the actual maximum stresses in the diagonals or verticals, neither does it give the position of reversal of stress.

The stress in D_1 is 7.92 tons tension and depends on the shear. The shear is maximum when the reaction is maximum. It is now essential to test whether the panel-point loading worked out from the greatest bending moment will give the correct value for the stress in the diagonal when the shear is greatest.

Suppose an 8-ton axle is over V_1 and the rest of the bridge fully loaded, then allowing a factor of 2 for live load and using four girders, the load on one will be as in *Fig. 8A*.



Taking a section xx and adding the vertical forces.

$$+V_1 + R_1 - \frac{1}{8} = 0.$$

R_1 under loading in *Fig. 8A* is 6.73 tons.

$$\therefore V_1 = -6.605 \text{ tons.}$$

Whence $D_1 = +9.313 \text{ tons.}$

The actual maximum stress in D_1 is then 9.313 tons compared with 7.93 tons found by the panel loading derived from the bending moment. A factor of 2 has been used in reducing live load to equivalent dead load. Suppose in calculating the shear a factor of $\frac{3}{2}$ had been used. Then with the loading in *Fig. 8A*

$$V_1 = -4.95. \quad D_1 = +6.98.$$

Therefore, if $\frac{3}{2}$ is a large enough factor for reducing the live to equivalent dead load, the stresses shown in *Fig. 8* are big enough for all conditions of loading.

It has already been stated that in the curve giving the distributed loads derived from the maximum bending moment a factor of 2 has been used to convert live to dead load. A factor of $\frac{3}{2}$ is really good enough but by using the factor 2 the difficulty of shear due to a train of concentrated loads is surmounted except on very short spans. It is true that in using this factor of 2 the booms are made too large but the booms are made up as a rule of several planks bolted together. In rough work in the field the booms will not be worked out to the nearest square inch but will be made up of a number of the largest planks at hand.

If the section of timber for the booms will not work in nicely with the theoretical stress it is quite safe to cut down the stress in the booms.

The object of the curve is to provide a quick method of getting the panel-point loads and is a compromise so that by using it all members in a girder will be safe.

From the N girder shown in *Fig. 8* many other forms can be evolved. Four girders as in *Fig. 8* will carry the required load. By cross-bracing we obtain a girder where there is no reversal of stress. This is shown in *Fig. 9*, the stresses being given in pounds. By halving the stresses in *Fig. 9* we obtain *Fig. 10*. Eight girders stressed as *Fig. 10* will carry the required load. By taking two girders stressed as *Fig. 10* and placing them one behind the other so that they overlap by half a panel length, fuzing them together, and adding the stresses in the members where they overlap we obtain a girder stressed as in *Fig. 11*.

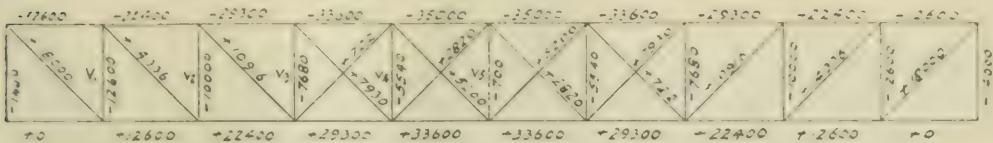
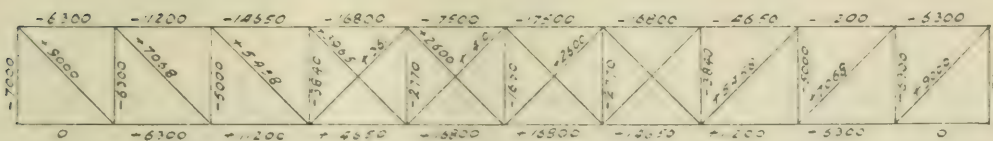


FIG. 9.



Stresses in Lbs. Tension = +
Compression = -

FIG. 10.

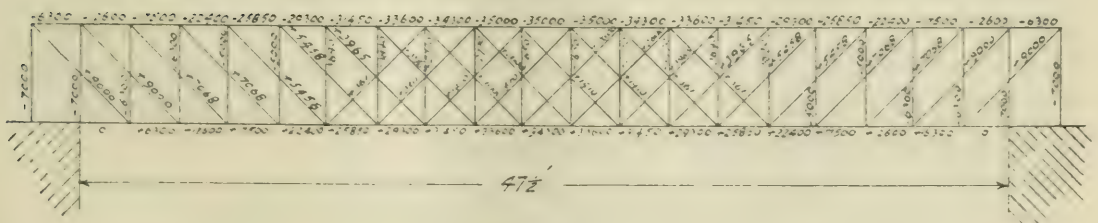


FIG. 11.

Each girder as in *Fig. 11* is twice as strong as the girder in *Fig. 10*. Therefore four girders as in *Fig. 11* will take the load. This form of girder is very easy to make if steel ties are available also since the verticals are close together local bending between the verticals is reduced. The girder must be supported as shown. This is important or the girder will fail. By supporting thus the span is reduced by 2 ft. 6 in. or half the length of a panel.

Returning to *Fig. 8*. Suppose the diagonals sloped the other

way. *Fig. 12* gives the stresses in a girder so constructed, *Fig. 12* being as strong as *Fig. 8*. Combining *Fig. 8* with *Fig. 12* and halving all the stresses we obtain *Fig. 13* which is as strong as *Fig. 8*. On combining *Figs. 8* and *12* the stresses vanish in all the verticals except the end ones, as we add the stresses in the members where they coincide. In the completed girder *Fig. 13* verticals are put in to take half the panel-point load.

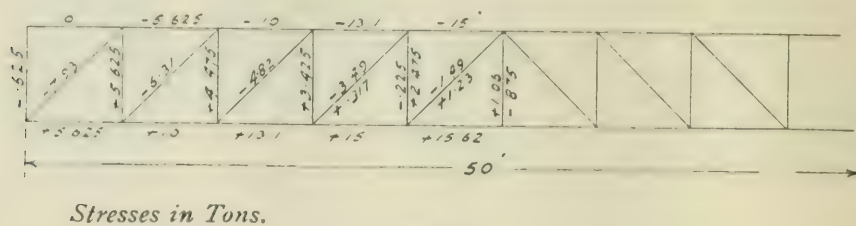


FIG. 12.

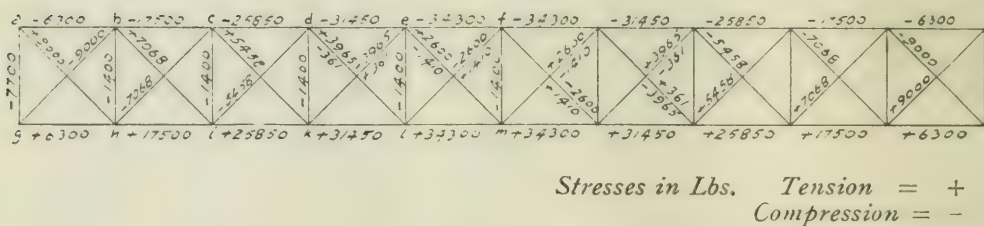


FIG. 13.

Now take *Fig. 13* and resolve the verticals along the sloping members. Next split *Fig. 13* into its two component girders, *Fig. 14* and *Fig. 15*. By a process of interpolating we can obtain two girders shown in *Fig. 16* and *Fig. 17*. Fuzing together the four girders in *Figs. 14, 15, 16* and *17* and halving the stresses throughout we obtain *Fig. 18* which is as strong as *Fig. 8*.

Therefore four girders as in *Fig. 18* will carry the load we require. The maximum stress in the top and bottom boom remain as in the original N girder shown on *Fig. 8*. In *Fig. 18* the end post is under bending stress besides direct compression.

From the original N girder nine others have been evolved, of these nine girders only four are useful in practice. These are *Fig. 9*, *Fig. 11*, *Fig. 13* and *Fig. 18*.

From *Plate III*. any form of girder can quickly be arrived at between the spans of 30 ft.—98 ft.

For the purposes of practically illustrating joints, etc., the four girders stressed as in *Figs. 9, 11, 13* and *18* will be worked out in detail.

CALCULATIONS FOR A GIRDER STRESSED AS IN *Fig. 9*.

Booms.—The greatest stress in the top boom is 35,000 lbs. compression. Assuming that the working stress of timber in compression is 1,200 lbs. per square inch, the size of the boom will be $\frac{35000}{1200}$ square inches = 30 square inches approximately. The largest scantling available is 9 in. $\times$ 3 in. and the booms will be made up of this 9-in. $\times$ 3-in. bolted together. This will entail boring holes in the wood and will reduce its working section. If however the bolts fit the bolt holes tightly they will not weaken the compression boom but the area of holes bored in the tension boom must be deducted from its working area. The working section of a 9-in. $\times$ 3-in. is 27 square inches. One 9-in. $\times$ 3-in. is therefore a shade too small for the boom in theory, but in practice a 9-in. $\times$ 3-in. would be all right. The boom must be jointed to get a 50-ft. length. This can be done either with iron fishplates or else with a cover plate of wood having the same sectional working area as the 9-in. $\times$ 3-in. boom. The use of iron fishplates is not recommended. By running the roadway on the top boom direct, local bending moments are set up between the panel points. If the boom is stressed up to its safe limit by direct compression it will fail under the local bending moment. With a light boom, in this case a single 9-in. $\times$ 3-in. the girder will be very whippy and so difficult to launch, besides this it will not be easy to make the joints between the steel diagonals and a single 9-in. $\times$ 3-in.

It is much better to use a continuous wood cover plate. In this instance using a 9-in. $\times$ 3-in. boom with a continuous 9-in. $\times$ 3-in. cover plate, the available section all through the top boom is 54 square inches. This will be obviously quite safe in direct compression and will leave a margin to resist the local bending moment.

Local Bending Moment.—The maximum local bending moment will be when a hind axle of the lorry is between two panel points. Using a factor of 2 for live load the maximum local bending moment is that produced by a partially distributed load of 4 tons on the centre of a 5-ft. span.

The 5-ft. span may be taken as a built-in beam and the load is distributed to a certain extent. This distribution will depend on the breadth of the members of the wood floor.

The actual bending moment will vary between $\frac{WL}{8}$ and $\frac{WL}{24}$ depending on the extent the load is distributed and the firmness of the fixing of the beam.

Suppose $\frac{WL}{12}$ is taken as the bending moment.

Then
$$\frac{M_F = 2240 \times 4 \times 60}{12} = 44800 \text{ inch-lbs.}$$

The boom is made up of two 9-in. $\times$ 3-in. deals. One of these 9-in. $\times$ 3-in. takes up the direct compression when it is at its maximum. Therefore one 9-in. $\times$ 3-in. is left to take up the local bending moment.

The moment of resistance of a 9-in. $\times$ 3-in. is

$$\frac{1}{6}rbd^2 = 200 \times 3 \times 81 = 48600 \text{ inch-lbs.}$$

This is greater than the maximum local bending moment and is therefore safe.

This is on the assumption that no boom joints come in the panels where the boom stress is greatest. The boom joints must be placed either over a vertical or between two panels where the boom stress is not at a maximum. The two portions of the cover plate or boom that take the bending moment may be considered at a joint to be cantilevers.

The tension boom will consist of one 9-in. $\times$ 3-in. with a continuous 9-in. $\times$ 3-in. cover plate. The joints in the tension boom will be made with $\frac{3}{4}$ -in. round iron bolts. The holes bored for these bolts will weaken the boom and its working section will now be

$$8\frac{1}{4} \times 3 = 24.7 \text{ square inches.}$$

The working stress for timber in tension is 1,600 lbs. per square inch.

Therefore the tension boom will take $24.7 \times 1,600 = 39,520$ lbs. and is quite safe.

Joints in the Boom.—All joints should be kept away from the centre of the boom when the stresses are greatest. The strength of the boom depends on the strength of the joints.

In Fig. 19 the strength of the boom in tension is equal to the strength of one 9-in. $\times$ 3-in. in tension provided the joint fastenings are good. In the tension boom under consideration, the greatest stress is 33,600 lbs. tension.

One $\frac{3}{4}$ -in. bolt passing through DD (Fig. 19) can safely take a pull of $\frac{3}{4} \times 3 \times 1200$ lbs. = 2700 lbs. before it begins to bear into the wood.

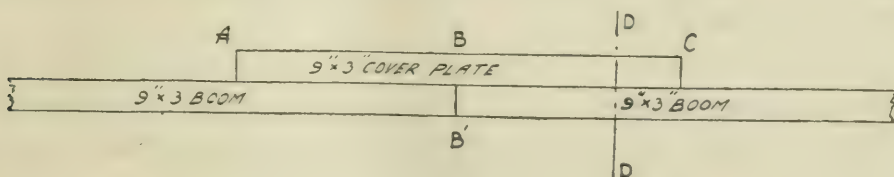


FIG. 19.

Therefore the number of bolts between A and B and B and C is $33600 \div 2700$ (the strength of one bolt) = 12 bolts (about).

Therefore 12 $\frac{3}{4}$ -in. bolts must be put on each side of every joint to make the tension boom safe. The same number will do for the compression boom. It is not always easy to find or to improvise nuts and bolts such as would be wanted for the booms. A quick way of making up the booms is to use plain round iron. Holes $\frac{1}{8}$ in. less in diameter than the round iron are bored in the booms and the round iron bars are cut to length and driven through. Bolts with nuts and washers are of course the best as by using them the members of the boom are drawn close together and friction strengthens the joint considerably. Besides this the nuts and washers prevent the different members of the compression boom opening out when the stress comes on.

The plain round drift bolts with no heads are quite good enough however if the booms are bound with hoop iron every few feet. This will effectually keep the booms from opening out when under compressive stress.

The booms will be made up as shown in *Plate IV*. A space of 3 in. is left between the members for the diagonals to pass through. At the junction of the verticals with the boom this space has 9-in. $\times$ 3-in. filling pieces.

Verticals.—The greatest stress in a vertical is 6.3 tons while the length of the verticals is 4 ft. 3 in. The ends may be considered fixed by the wooden cleats shown in *Plate IV*.

From the curve for struts, *M.E.*, Part III.A, a suitable strut can be looked out. A 9-in. $\times$ 3-in. will just do for the end strut and will be used all through. This saves cutting and does not add much to the weight.

Diagonals.—Assuming round bar steel to be available, and that the safe working stress for steel in tension is 7.5 tons per square inch we can arrive at the following facts :—

A $1\frac{1}{2}$ -in. round steel bar cut with a Whitworth screw thread takes	21,504 lbs. tension.
A $1\frac{3}{8}$ -in. " " "	" 16,800 lbs. "
A $1\frac{1}{4}$ -in. " " "	" 13,440 lbs. "
A 1-in. " " "	" 8,072 lbs. "

Suiting these bars to the stresses in the diagonals the result shown in *Plate IV*. is obtained.

Jointing of Diagonals.—The members of the boom are opened out as shown in *Plate IV*. and 9-in. $\times$ 3-in. packing pieces inserted. The steel diagonals come through these packing pieces and will be nutted up against hardwood blocks attached to the top of the boom. A steel plate will be placed between the nut and the hardwood block.

Calculation for the Wood Blocks.—There will be a tendency for the blocks to fail either by slipping along the top boom or if they can't do this to shear along their own length. There is also a danger of

the wood blocks failing through the washers under the nuts bearing into them. Considering the diagonal under greatest tension. The resolved part of this tension along the boom is 12,600 lbs. Therefore the fastening of the blocks to the boom must be able to resist this. A $\frac{3}{4}$ -in. coach screw will take a pull of 2,100 lbs. (this figure is taken from *M.E.*). Therefore the block must be held by six $\frac{3}{4}$ -in. coach screws, provided the coach screws do not fail by bearing into the hardwood block. To test this supposing the blocks are 9-in. $\times$ 3-in. oak. The safe resistance to bearing of a $\frac{3}{4}$ -in. screw through a 3-in. oak block is $\frac{3}{4} \times 3 \times 1600$ lbs. = 3600 lbs. This shows the screws are safe as far as bearing is concerned.

Therefore the block will be safe if six $\frac{3}{4}$ -in. coach screws are used. The screws must be staggered or they will weaken the boom and are arranged as shown in *Plate IV*. The number of screws in each block may be reduced by one as they get towards the centre. The blocks may shear along the grain. The resistance of oak to shear along the grain is 450 lbs. per square inch.

The number of square inches required is therefore $\frac{12600}{450} = 28$.

The size of the blocks is 9 in. $\times$ 3 in. $\times$ 15 in. which gives more than the area required.

The resistance of oak to bearing is 1,600 lbs. per square inch. The direct bearing stress is 18,000 lbs., this must be distributed over $\frac{18000}{1600} = 11$ square inches (about).

The surface of the block is 36 square inches and the stress is distributed over this by a $\frac{1}{2}$ -in. steel plate. Therefore the diagonal joints are all safe if made as shown in *Plate IV*.

CONSTRUCTION OF THE GIRDER STRESSED AS IN FIG. II.

Booms.—The booms have the same maximum stresses as in the previous case and are made up of two 9-in. $\times$ 3-in. one acting as the boom and the other as a continuous cover plate. Twelve bolts are put in on each side of every joint and in this case no space is left between the booms.

The booms are shown in *Plate V*.

As the booms in the previous case were safe against local bending they will obviously be safe here too as the panels are 2 ft. 6 in. long against 5 ft. in the other instance.

Verticals.—The maximum stress is 3.15 tons compression. From the curve for compression members, *M.E.*, Part III.A, *Plate XIII.*, one 6-in. $\times$ 3-in., 4 ft. 3 in. long, takes about 2 tons. For the two end verticals at each end use two 6-in. $\times$ 3-in. nailed together. For the remaining verticals use 6-in. $\times$ 3-in. These will be quite strong enough as the ends are "fixed" by wooden blocks shown in *Plate V*.

Ties.—Suppose that flat bar steel is available. The greatest stress in a tie is 9,000 lbs. while the working stress of steel in tension is 16,800 lbs. per square inch. The working section of each tie must therefore be $\frac{9000}{16800} = .54$ square inches. Using one brace on each side of the boom for every tie it is clear that each brace must have a working area of .27 square inches.

The ties will be fixed to the booms by $\frac{5}{8}$ -in. coach screws. From the table of holding power of nails, etc., each $\frac{5}{8}$ -in. coach screw can take a pull of 1,650 lbs.

Each brace of the end diagonals has a tension stress of 4,500 lbs.

Therefore the number of coach screws required at the end of each brace is $\frac{4500}{1650} = \text{three screws}$.

Suppose 2-in. $\times$ $\frac{1}{4}$ -in. flat bar steel is available, this when bored for a $\frac{5}{8}$ -in. bolt has a working area of .345 square inches.

Therefore the end diagonals will be safe if made up of 2-in. $\times$ $\frac{1}{4}$ -in. steel fixed at each end with three $\frac{5}{8}$ -in. coach screws. Towards the centre the tension in the diagonals gets smaller and so the number of coach screws and the size of the ties may be reduced. In practice the ties would be kept the same all through as it would not be worth while to cut up the steel bars into smaller sizes. The number of coach screws might be reduced as shown in *Plate V.*, which gives the details of the completed girder.

CONSTRUCTION OF THE GIRDER SHOWN IN FIG. 13.

Booms.—These will be the same as in the previous cases but for the purposes of constructing the joints will have a space of 1 in. left between the boom members. This is illustrated in *Plate VI.*

Tension Members.—Each tension member will be made up of two pieces of timber one fixed on each side of the boom. Considering the member *ah* in *Fig. 13.* The greatest stress is 9,000 lbs. tension while the working stress of timber in tension is 1,600 lbs. per square inch. Therefore the working section of each member of the diagonal *ah* is

$$\frac{9000}{2} \div 1600 = \text{say } 3 \text{ square inches.}$$

Allowing 1 square inch to be cut out of these members for the fastenings two 4-in. $\times$ 1-in. battens would do for the diagonal *ah*.

If thin battens are used they may fail by the metal fastening either bearing into them or shearing through the ends. The size of the wood tension members will therefore depend on the fastenings and not on the sectional area of the members. Suppose that $\frac{5}{8}$ -in. coach screws are used for fastening the diagonals to the boom. Each screw will take a pull of 1,600 lbs. and since the stress in each half of the tension member is 4,500 lbs. three coach screws must be used

at each end. The timber ties must be so arranged that they will be strong enough to resist the tendency of the coach screws to bear into them or to tear out at their ends.

Let x in. be the thickness of one tension member, then the resistance to bearing against a $\frac{5}{8}$ -in. coach screw is $x \times \frac{5}{8} \times 1,200$ lbs. Equating this to the strength of one coach screw we have

$$x \times \frac{5}{8} \text{ in.} \times 1,200 = 1600.$$

Whence $x = 2.15$ in. Say 2 in.

Therefore the ties must be at least 2 in. thick.

To test how far from the end of a 2-in. tie it is safe to put a $\frac{5}{8}$ -in. screw. Let l in. be this distance. The working stress of fir in shear along the grain is 150 lbs. per square inch. If the screw is to be as strong as the 2-in. tie we obtain the equation.

$$l \times 2 \times 2 \times 150 = 1600.$$

$$\therefore l = 2\frac{3}{4} \text{ in. (about).}$$

Therefore no screw must be less than $2\frac{3}{4}$ in. from the end of the 2-in. ties.

The diagonal ah will consist of two 4-in. $\times$ 2-in. fir battens one on each side of the boom, each tie being fastened by three $\frac{5}{8}$ -in. coach screws at each end. A breadth of 4 in. is taken to allow plenty of room for the screws. The completed joint is shown in *Plate VI*.

The diagonal bi will be the same as ah . The diagonal ck could be reduced in size but to prevent extra cutting is kept the same size. Two $\frac{5}{8}$ -in. screws only are used for the diagonals ck and dl . The diagonal dl being made up of two 4-in. $\times$ 2-in. battens will safely take the reversal of stress of 361 lbs. The diagonal em has to take 2,600 lbs. tension and may have to take 1,410 lbs. compression. A 4-in. $\times$ 3-in. batten with a $1\frac{1}{2}$ -in. shoulder cut to fit under the boom and held by two $\frac{5}{8}$ -in. coach screws will be amply strong enough to take these stresses. One 4-in. $\times$ 3-in. will take the stress but in this case there is reversal of stress that will cause the wood to tire. A large factor of safety is wanted. In this case using two 4-in. $\times$ 3 in. there is a factor of safety of about 10.

Diagonals in Compression.—The diagonal gb has a stress of 9,000 lbs. compression. Its length is 6 ft. From the curve for struts a 7-in. $\times$ 3-in. is found strong enough. This 7-in. $\times$ 3-in. strut will have an inch tenon cut at each end. The tenon will be slipped between the two 9-in. $\times$ 3-in. deals forming the boom. Filling pieces will be added between each joint and the tenons will be held by 6-in. nails put through the boom. All the diagonals will be made of 7-in. $\times$ 3-in.

shouldered the same way as the diagonal *gb*. The diagonals *ek*, *lf*, will be held by six 6-in. nails, this will enable them to take up any tension that can come on them. These joints are shown on *Plate VI*.

Verticals.—The greatest stress is in the end vertical and is 3.5 tons compression. Two 7-in. $\times$ 3-in. struts will take this while one 7-in. $\times$ 2-in. is used for the other verticals.

The girder is made up as shown in *Plate VI*.

There is no need to repeat the calculations for joints, etc., in the case of a girder stressed as in *Fig. 18*. The completed girder and joints is shown on *Plate VII*.

Plate VIII, shows a girder capable of taking the stresses shown in *Fig. 8*. The calculations for the joints, etc., of this girder will not be gone into in detail. The diagonals are made of telegraph wire having a breaking load of 1,260 lbs. The joints between the wire and the rails is clearly shown on *Plate VIII*. The number of steel bars used depends on the bearing stress between the steel bars and the thin webs. With larger rails fewer bars could be used. The local bending moment between panel points will stress the steel up rather high and a larger section for this reason is advisable too. The rail joints should be kept over a vertical. Another point is that unless the rails are well fixed to the roadway they will tend to fail by buckling sideways under direct compression. This tendency is stopped if the girders are all firmly held to a heavy roadway. One of these rail girders was tested with a 6-ton central load. It just took this but any further load would have crippled the girder by side buckling between the panel points. This as stated above could not happen with four girders in a bridge with the rails braced by the roadway. This form of girder takes longer to make than a wooden one as there is a good deal of drilling required.

So far no consideration has been given to wooden plate girders. It is clear that if the process of subdivision and interpolation (by which the stresses on *Fig. 18* were obtained) was carried on indefinitely we should ultimately arrive at a girder with light members so close together that they would form a solid web and would in fact be a plate girder. In *Fig. 19* the members sloping upwards from left to right are in compression and those from right to left in tension. This of course is the case in a plate girder as well. The quickest method of getting out a plate girder is by the method indicated in *M.E.*, III.B. First find the maximum bending moment and the maximum shear under a train of loads. From the bending moment get out the size of the booms and from the shear get out the size of the web. This web is calculated on the assumption that the web will fail by directly shearing through the fibres of the wood in the web. This would never happen in practice as the

web members would fail by buckling or tearing away from the fastenings at the boom. To avoid this *M.E.*, Part III.B, recommends the use of verticals and stiffening blocks between the web. By doing this a girder that will take the calculated load is obtained.

There appears to be no need to investigate exactly what stress is in each member of the plate girder though this can easily be done from the previous reasoning and treating the plate girder as a multiple lattice girder. An example of a plate girder is worked out on the principle shown in *M.E.*, Part III.B.

Taking the same 50-ft. gap with the same loading

$$M_{\text{ff}} \text{ due to equivalent dead load} = 7,106,400 \text{ inch-lbs.}$$

$$M_{\text{ff}} \text{ due to dead load} = 1,680,000 \text{ inch-lbs.}$$

$$\text{Max. bending moment} = 8,786,400 \text{ inch-lbs.}$$

Using four girders the M_{ff} in each = 2,196,600 inch-lbs.

The formula for the size of boom is

$$M_{\text{ff}} = rAd,$$

where d is the effective depth in inches,

A is the area of one boom in square inches,

r is the working stress of timber in lbs. per square inch.

In this case $d = \frac{\text{span}}{14} = \text{say } 40 \text{ in.}$ while $r = 1,200 \text{ lbs. per square inch.}$

The equation is now

$$2,196,600 = 1200 \times A \times 40.$$

Whence $A = 45.76 \text{ square inches.}$

Therefore the booms will be made up of two 9-in. $\times$ 3-in. deals with a continuous 9-in. $\times$ 3-in. cover plate.

As before 12 $\frac{3}{4}$ -in. bolts will be wanted on each side of every joint.

To find the Thickness of the Web.—This depends on the maximum shear. The shear will be maximum when the bridge has as many lorries on it as possible with one of the heavy axles just on an abutment. The greatest shear in this case is 31.24 tons on the whole bridge or 7.81 tons on each girder.

The web will be of timber sloping from right to left on one side and left to right on the other.

The fibres of the timber in the web will be in shear diagonally. Timber takes safely in shear with the grain 150 lbs. per square inch

and across the grain 450 lbs. per square inch. Therefore in this case the mean of these two values *i.e.* 300 lbs. per square inch will be a correct value for the resistance of the web to shear.

The web has to take a shear of 7·81 tons.

If t is the thickness of the web in inches and the depth is 40 in. then

$$40 \times t \times 300 = 7 \cdot 81 \times 2240.$$

$$\therefore t = 1 \frac{1}{2}'' \text{ nearly.}$$

The web must therefore be made up of two thicknesses $\frac{3}{4}$ -in. battens, one layer on each side of the boom, one layer sloping upwards from left to right and one from right to left. The former being in compression and the latter in tension as shown in *Plate IX*.

Fastening the Web to the Boom.—The vertical shear is 7·81 tons in a 40-in. web. Since the horizontal shear is the same as the vertical shear at any point we can assume that the horizontal shear in the first 40 in. of the girder is 7·81 tons or 17,494 lbs.

Suppose that the web is held to the boom by 6-in. nails. Each 6-in. nail takes a working load of 200 lbs. Therefore the number of 6-in. nails required in the first 40 in. of the boom is $\frac{17494}{200} = 67$ nails. Half of these will be on one side of the boom and half the other, that is about $1 \frac{1}{4}$ in. apart on each side of the boom.

Stiffeners for the Web.—The thin $1 \frac{1}{2}$ -in. web must be stiffened. The actual position and size of these stiffeners can be worked out on the same lines as the stiffeners for a steel plate girder. For work in the field these calculations may be reasonably omitted, and the following method adopted which will give safe results:—

Verticals are put in between the booms at a distance apart of $\frac{2d}{3}$ in.

In this case since $d=40$ the verticals will be 2 ft. 3 in. apart near the ends but gradually widening out towards the middle. These verticals must be capable of taking a compression equal to the greatest reaction.

The length of the struts is 31 in. the compression is 7·81 tons.

Therefore the struts must be 9 in. $\times 2 \frac{1}{2}$ in., say 9 in. $\times 3$ in.

They will be fixed to the boom by hoop iron as shown on *Plate IX*.

In addition to these verticals blocks of wood are nailed between the web members to keep them stiff. The web members are nailed to the verticals. The whole girder is made up as shown in *Plate IX*.

Notes on Construction of Joints.—In no case is friction counted on as helping to strengthen any joint, therefore the expansion and contraction of the timber under climatic conditions does not matter. It is a good plan to put large washers under all nuts or coach screws, but the nuts should not be done up too tight or they push the washers

into the timber and so damage the fibres. A good deal of supervision is wanted during the making of these girders. The principal points to watch are :—

- (1). Start making the booms from the centre and see that there is no joint near the centre. The men if left to themselves always work from one end.
- (2). Make sure that no two bolts in the boom joint are in the same vertical plane and do not have a long line of bolts in the same horizontal plane as this leads to the timber splitting.
- (3). Stagger all fastenings when the diagonals meet the booms and keep the joint fastenings at a safe distance from the ends of diagonals.
- (4). Where members of a structure are in alternate compression and tension use a large factor of safety.
- (5). The values of resistance of timber to tension and compression used in these examples are for sound fir. If wood joists from houses, etc., are used they should be tested. All rusty nail holes, etc., in wood taken from buildings should be deducted when calculating for tension members.

Plate X. shows the section of a completed road bridge having inside and outside bracing. The latter is redundant. The whole bridge will be found to sway considerably until the roadway is spiked down, when the bridge becomes very rigid.

The thickness of the roadway depends on the spacing of the girders and the wheel track of the vehicles that pass over.

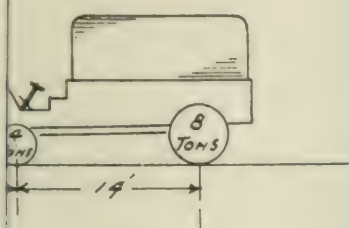
The maximum bending moment on the roadway when most unfavourably loaded should be worked out and the thickness of the roadway made such that it will resist this bending moment. Wheel tracks running along the length of the bridge give a smooth surface for the wheels to run on, are easily replaced when worn, save shocks on the bridge due to unevenness in the roadway and also save wear on the timbers forming the roadway. Crosspieces between these wheel tracks will prevent horses slipping about on the timber roadway in wet weather. The handrail should be strong and space left between the wheel guides and the handrail to allow foot passengers to keep clear of wheeled traffic.

The deflection of these 50-ft. girders under full load is very small while the permanent sett is inappreciable, except in the case of the girders shown in *Figs. 9 and 11*, where the deflection was from 2 to 3 inches, the permanent sett being from $1\frac{1}{2}$ to 1 inches.

GIVE THE MAX BENDING MOMENTS

ARRIES.

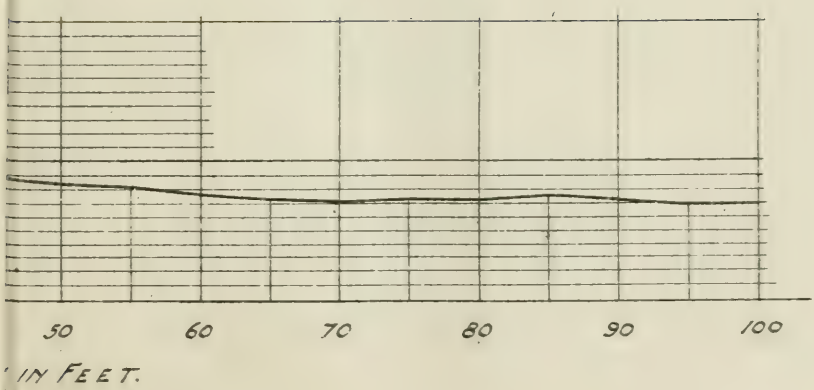
AINED BY MULTIPLYING THE LIVE LOAD
BY TWO.



TO EQUIVALENT DEAD LOAD FOR
THE FOLLOWING :-

FOOT RUN FOR SPAN X PANEL LENGTH
NUMBER OF GIRDERS USED.

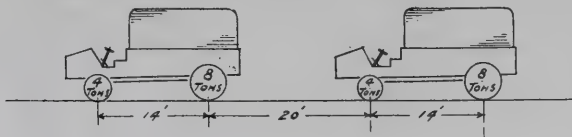
SPAN.	EQUIVALENT DEAD LOADS PER FOOT RUN.
5'	6.4 TONS.
10'	3.2 "
15'	2.132 "
20'	1.6 "
25'	1.28 "
30'	1.14 "
35'	1.03 "
40'	.932 "
45'	.886 "
50'	.846 "
55'	.820 "
60'	.764 "
65'	.742 "
70'	.718 "
75'	.738 "
80'	.750 "
85'	.756 "
90'	.750 "
95'	.744 "
100'	.744 "



GRAPH SHOWING LOAD PER FOOT RUN THAT WILL GIVE THE MAX BENDING MOMENTS UNDER A TRAIN OF THE HEAVIEST MOTOR LORRIES.

THE VALUES ARE EQUIVALENT DEAD LOADS OBTAINED BY MULTIPLYING THE LIVE LOAD BY TWO.

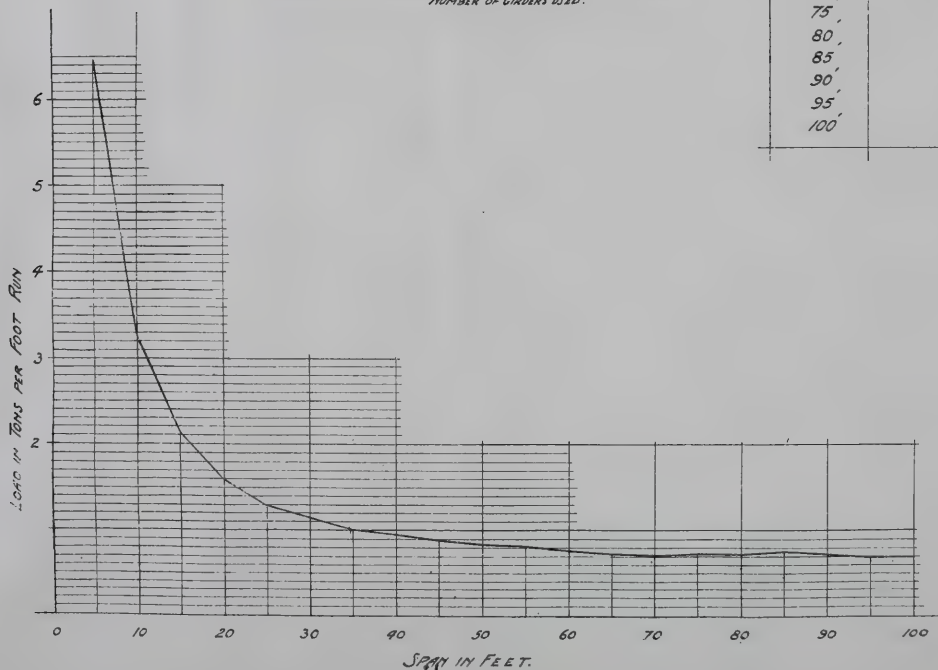
THE LOADING IS THUS :-



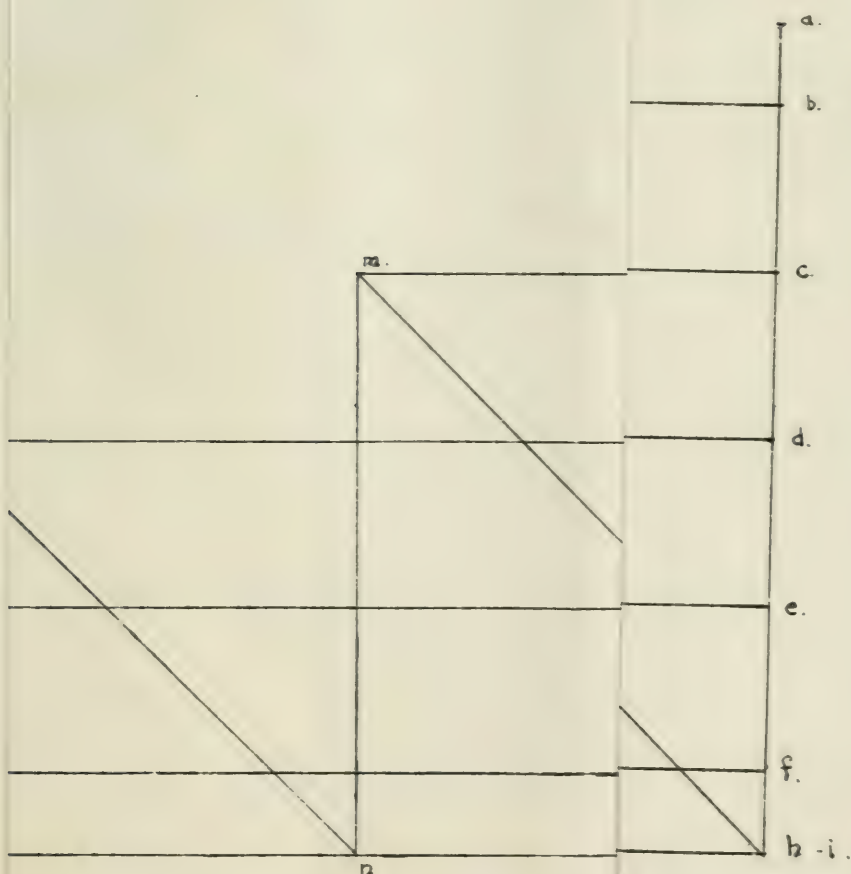
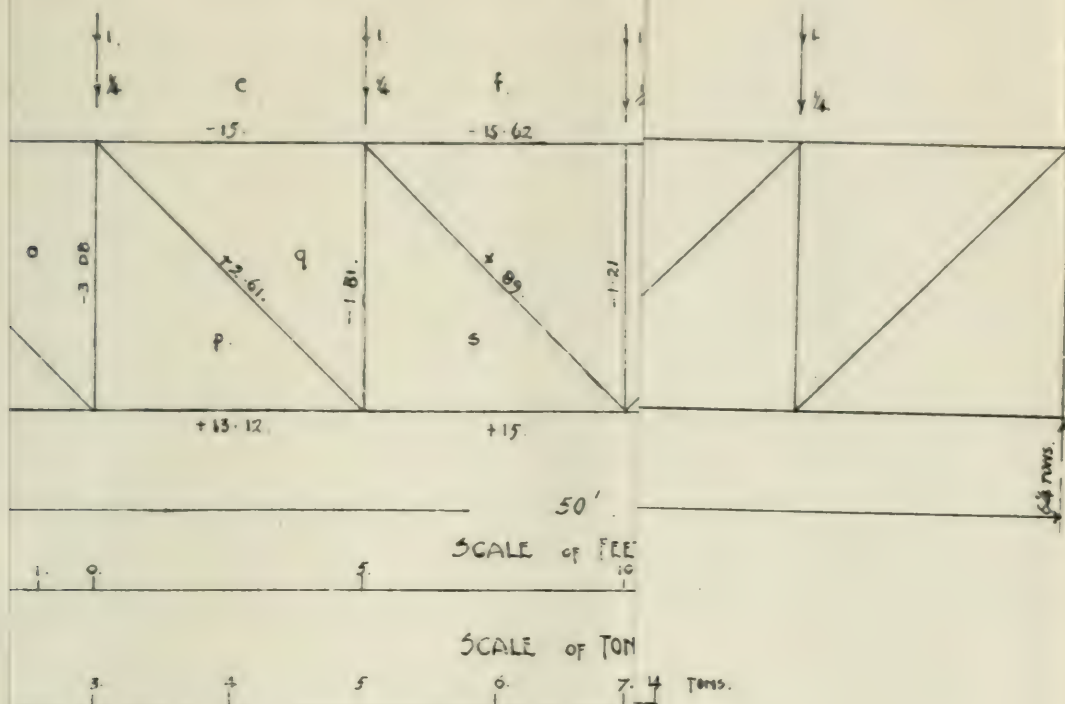
TO OBTAIN THE PANEL LOADS DUE TO EQUIVALENT DEAD LOAD FOR EACH N GIRDER IN A BRIDGE USE THE FOLLOWING :-

EQUIVALENT DEAD LOAD PER FOOT RUN FOR SPAN X PANEL LENGTH
NUMBER OF GIRDERS USED.

SPAN.	EQUIVALENT DEAD LOADS PER FOOT RUN.
5,	6.4 TONS.
10,	3.2 "
15,	2.132 "
20,	1.6 "
25,	1.28 "
30,	1.14 "
35,	1.03 "
40,	.932 "
45,	.886 "
50,	.846 "
55,	.820 "
60,	.764 "
65,	.742 "
70,	.718 "
75,	.738 "
80,	.750 "
85,	.756 "
90,	.750 "
95,	.744 "
100,	.744 "



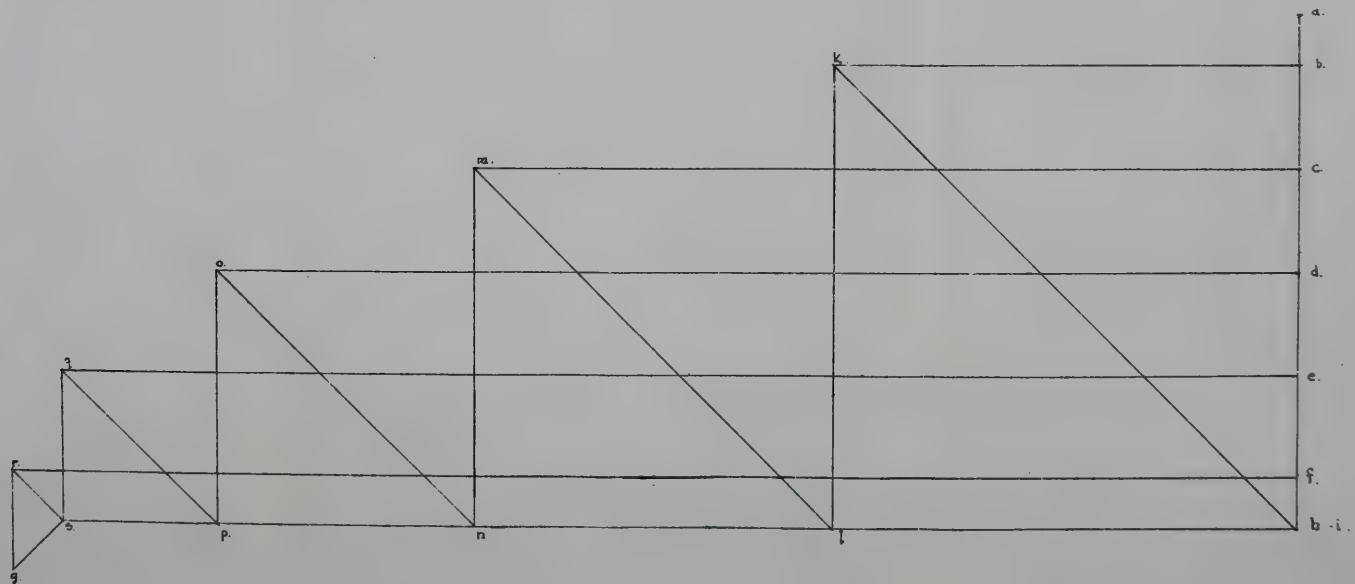
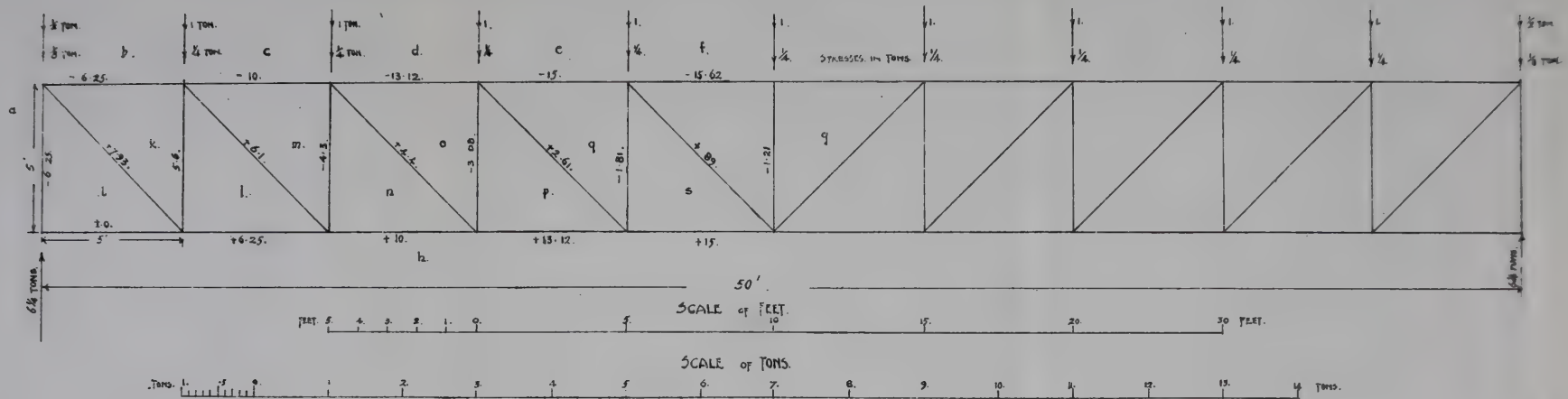
at Maximum Stresses for every Member
 II Load. Compare with Fig. 8, which



STRESSES IN A 50-FT. N GIRDER FOUND GRAPHICALLY.

PLATE II.

(Note:—These are not Maximum Stresses for every Member, but Maximum Stresses for each Member under Full Load. Compare with Fig. 8, which gives actual Maximum Stresses).



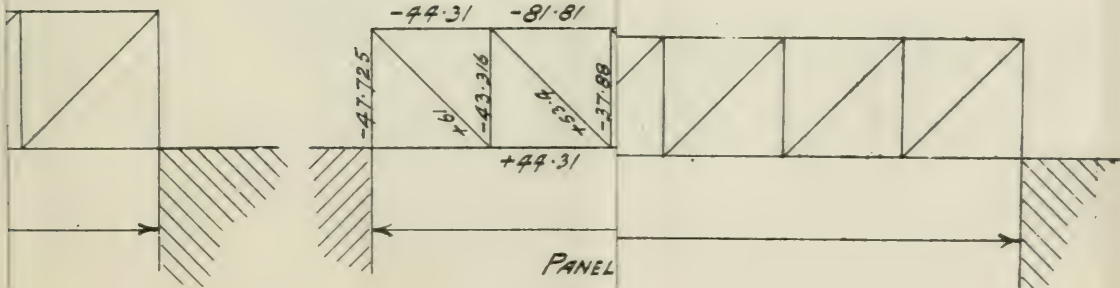
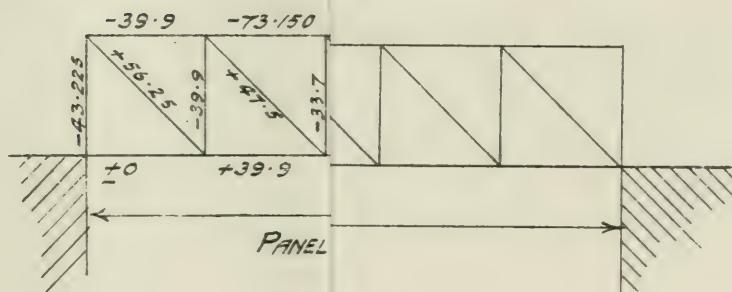
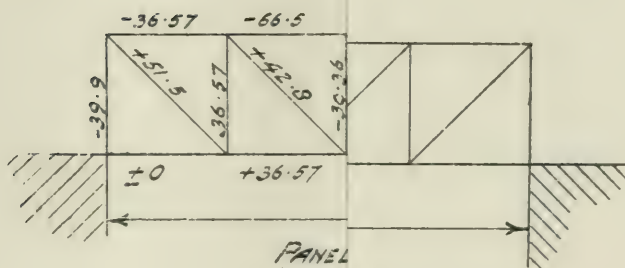
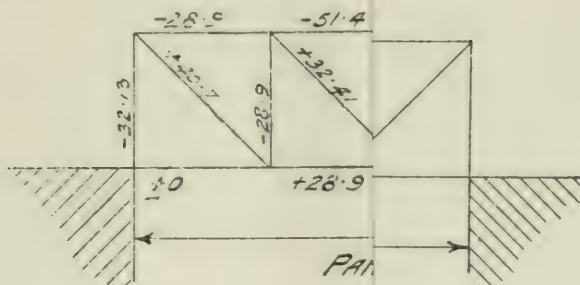
ON TOP BOOM, DESIGNED TO

PLATE III.

NOT RUN FOUND FROM CURVE.

NO

SES IN TONS. COMPRESSION SIGN - ve
TENSION " + ve



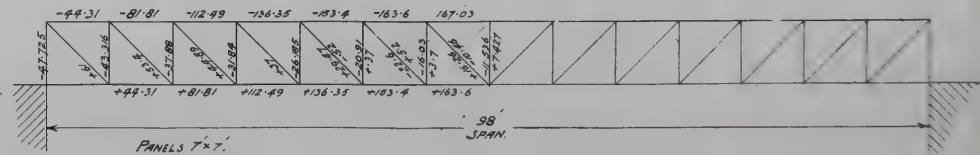
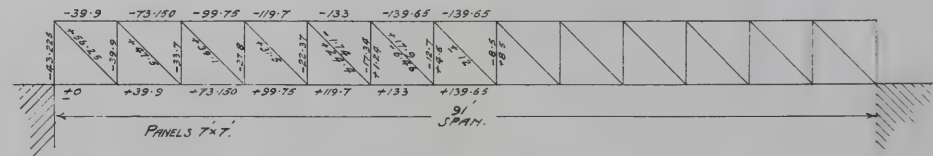
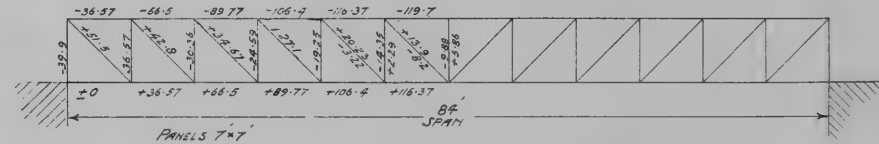
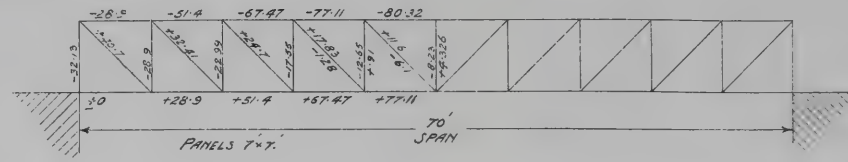
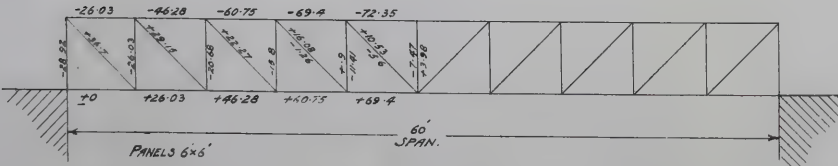
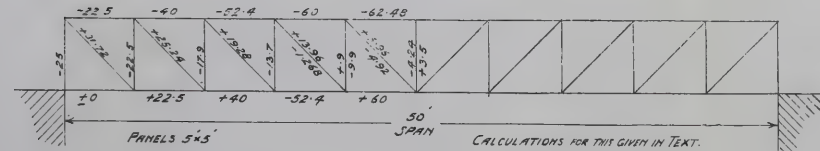
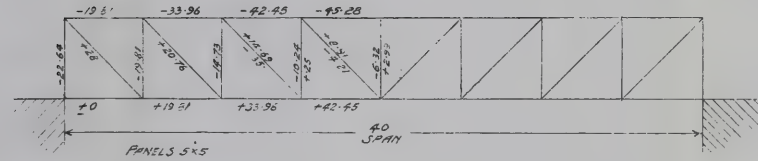
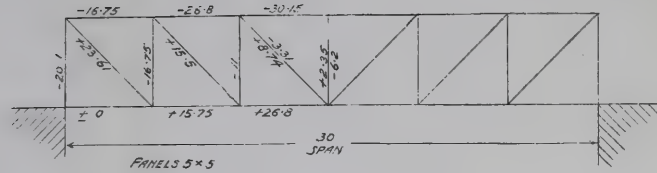
EQUIVALENT DISTRIBUTED DEAD LOAD PER FOOT RUN FOUND FROM CURVE.

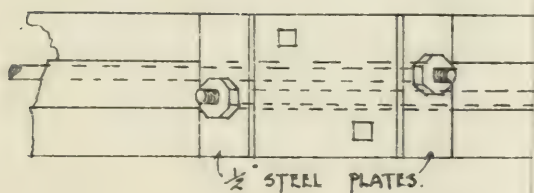
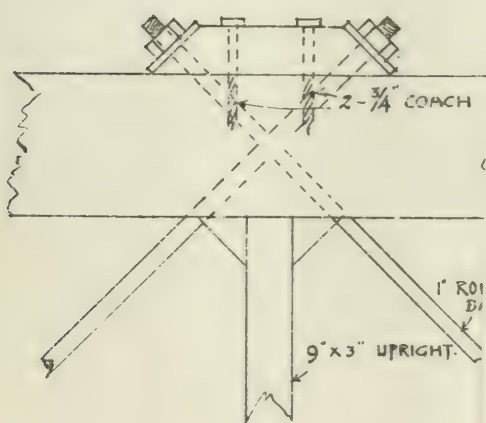
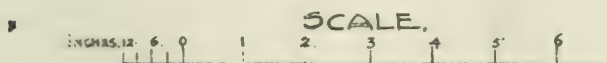
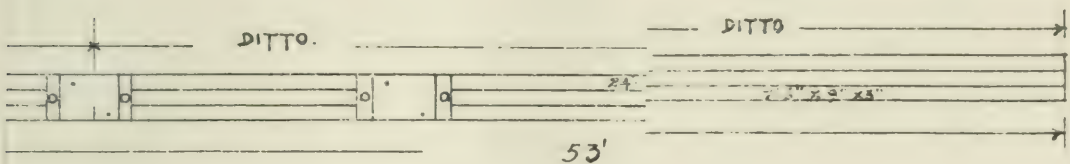
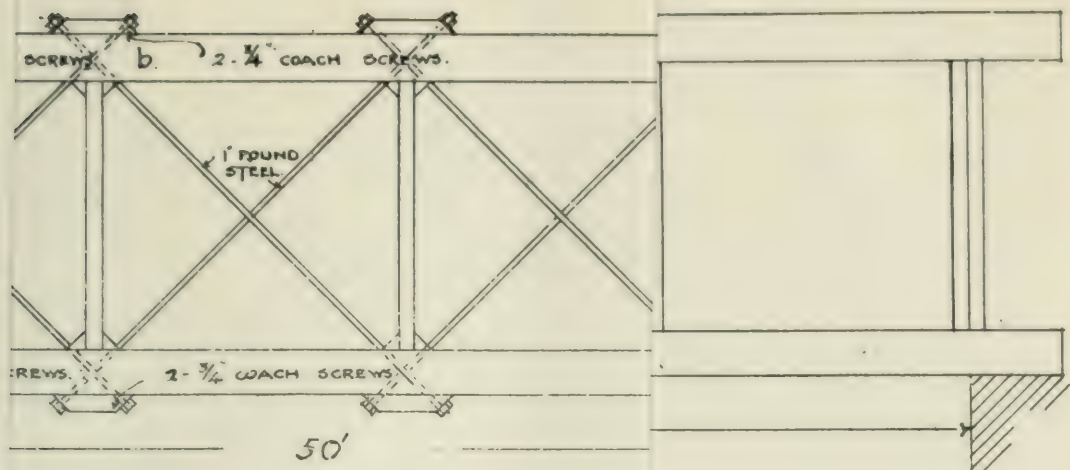
DEAD LOAD ASSUME $\frac{1}{8}$ TON PER FOOT RUN

ALL STRESSES IN TONS.

COMPRESSION SIGN -
TENSION " +

NOTE:- THE STRESSES GIVEN IN THIS PLATE ARE STRESSES IN THE WHOLE BRIDGE.

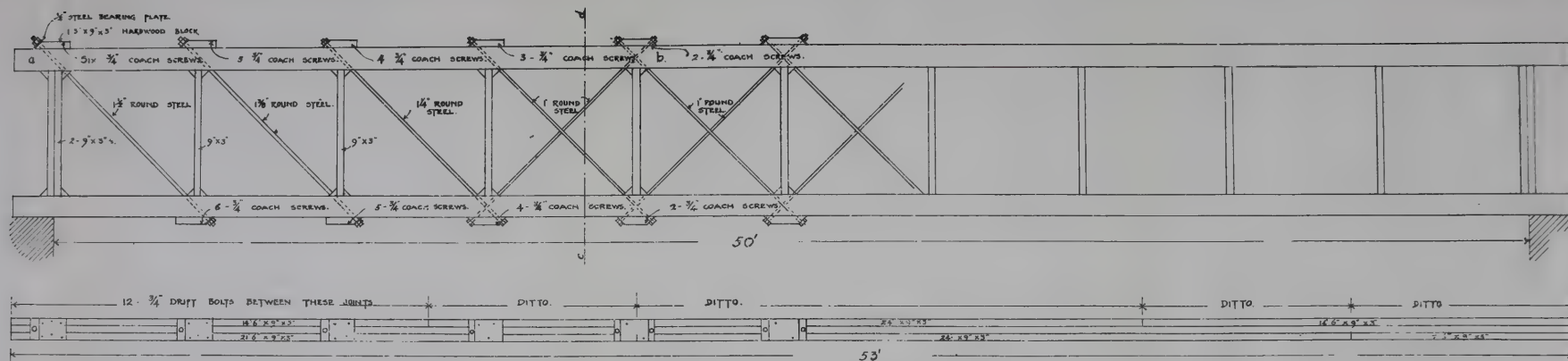
TO FIND THE STRESS IN ANY ONE GIRDER DIVIDE THE GIVEN STRESS BY
THE NUMBER OF GIRDERS IN THE BRIDGE.



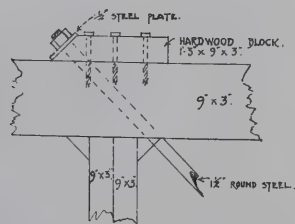
JOINT AT b.

FOUR SUCH GIRDERS TO CARRY A TRAIN OF LORRIES, ROADWAY ON TOP BOOM. STRESSES AS IN FIG. 9.

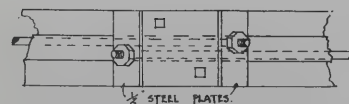
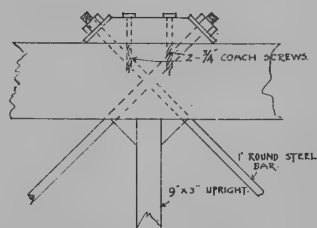
PLATE IV.



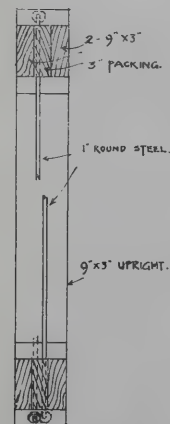
SCALE.
INCHES 0 1 2 3 4 5 6 7 8 9 10 FEET.



JOINT AT 'a'.



JOINT AT 'b'.

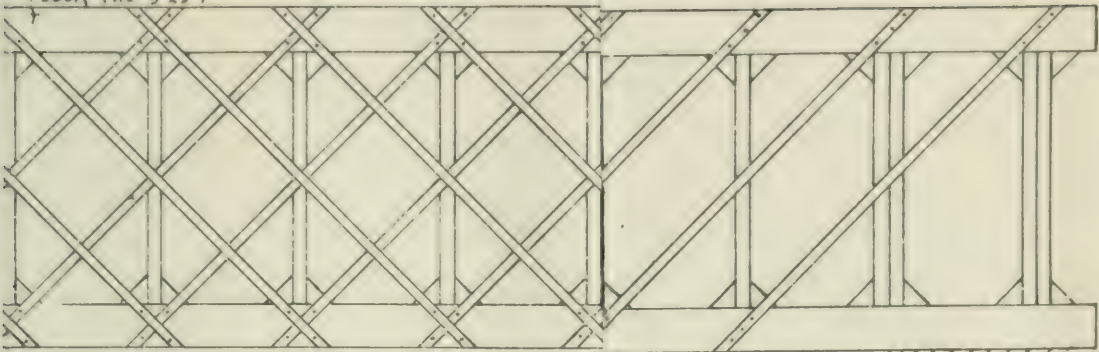


SECTION C-D.

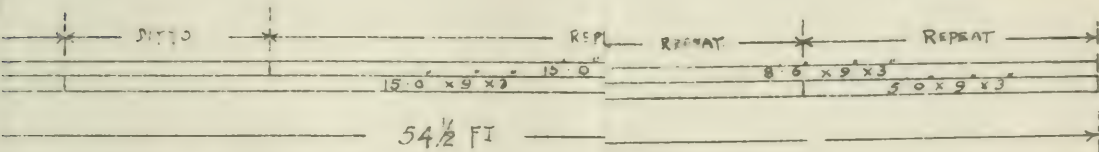
RDERS TO CARRY HEAVY M

PLATE V.

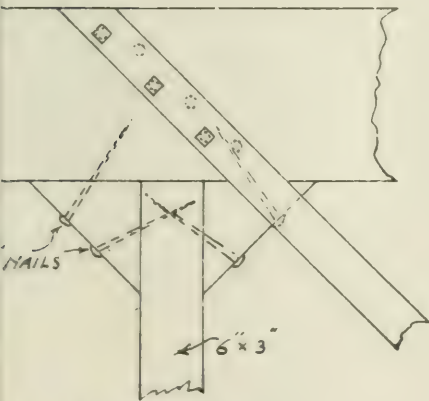
AT EACH END TO HAVE THREE
THE REMAINING BARS
VS
BOOM TWO 9"x3"



47½ FEET.

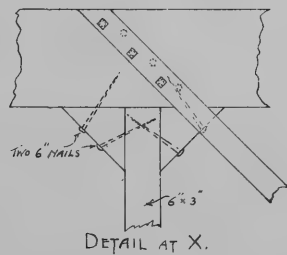


PLAN OF BOOM.

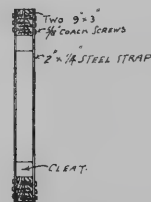


DETAIL AT X.

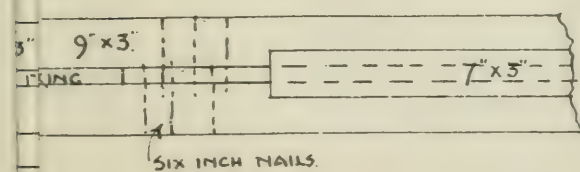
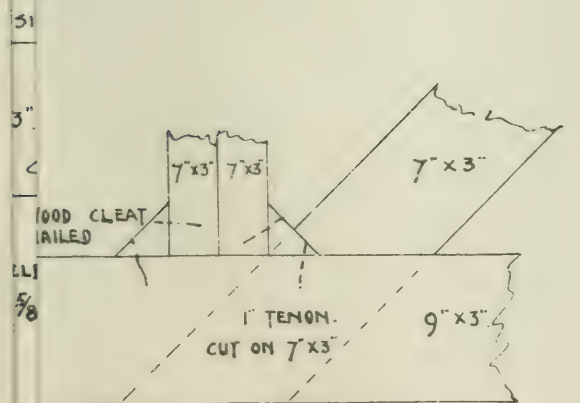
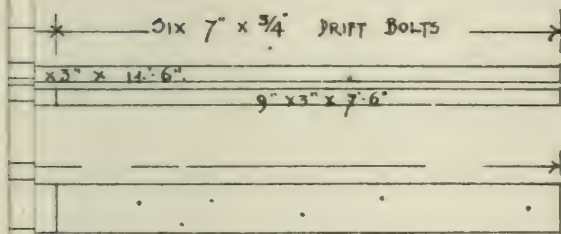
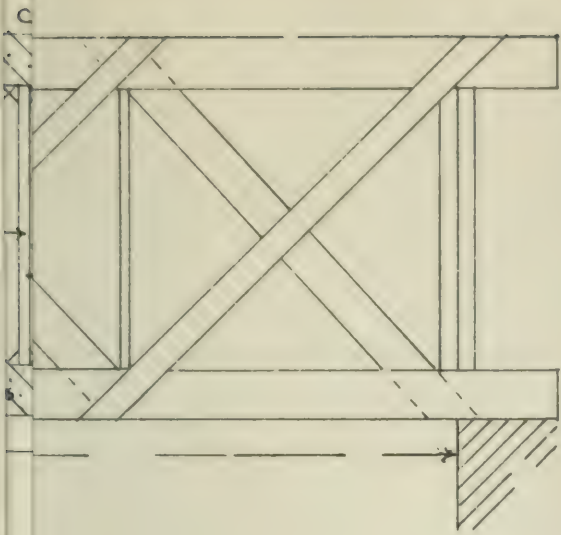
PLATE V.



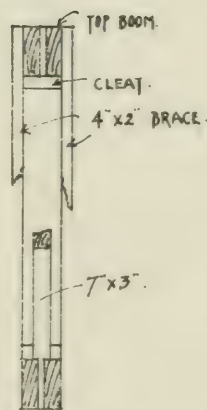
DETAIL AT X.



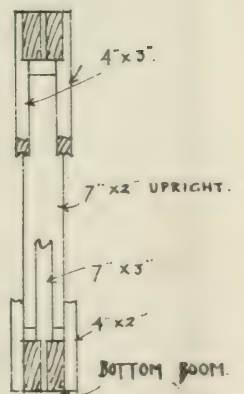
SECTION A.A.



PLAN & ELEVATION of JOINT AT g.



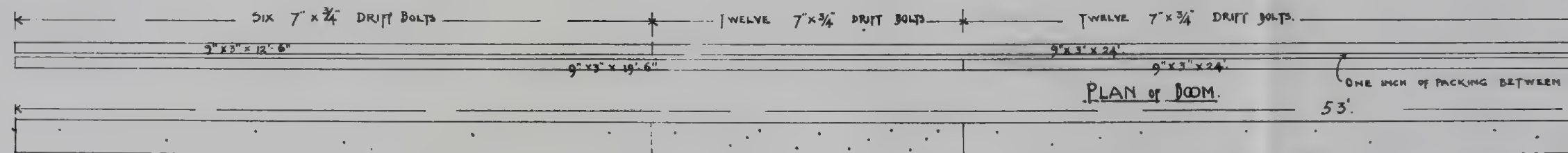
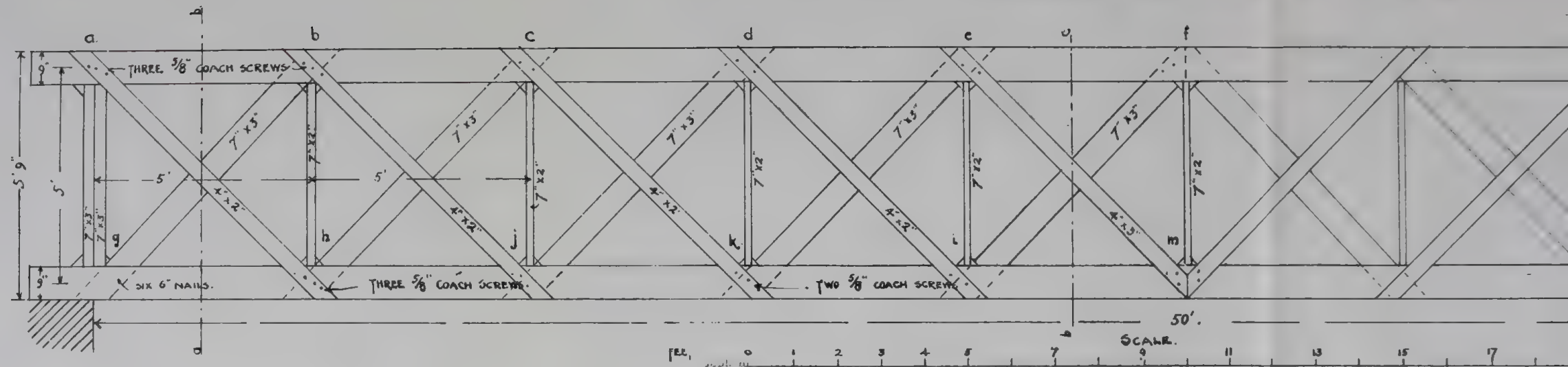
SECTION on a-b



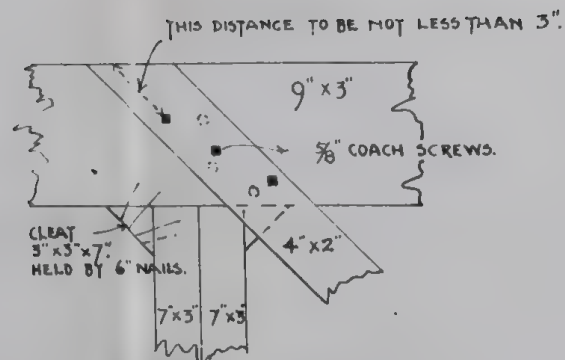
SECTION on b.c.

FOUR SUCH GIRDERS TO CARRY TRAIN OF LORRIES, ROADWAY

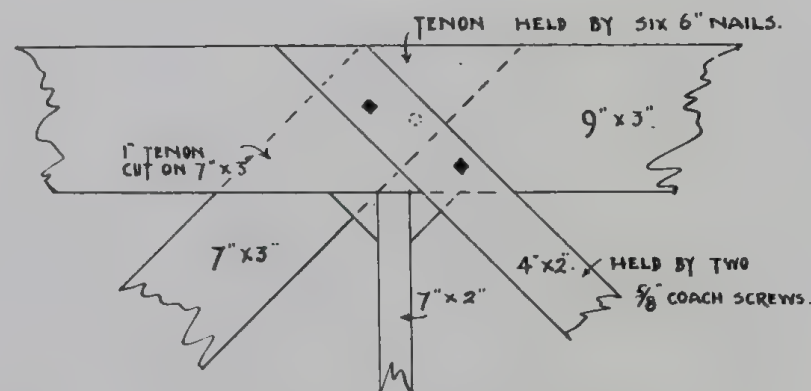
TOP BOOM. STRESSES AS IN FIG. 13.



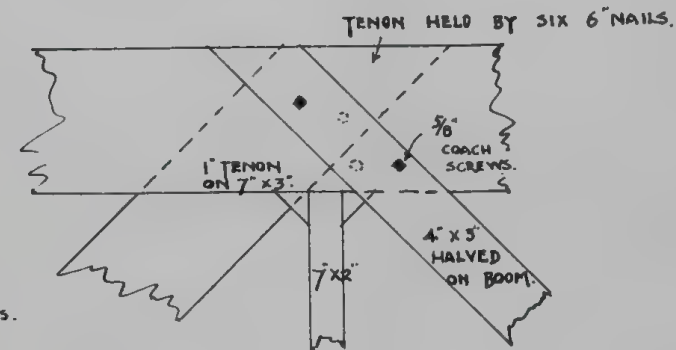
ELEVATION of BOOM, SHEWING ROUGHLY POSITION of D



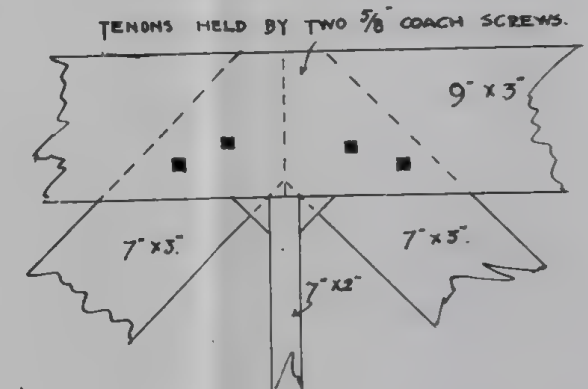
ELEVATION of JOINT at a.



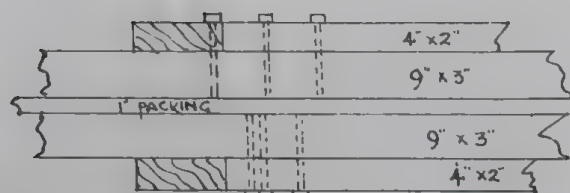
JOINT AT c, d, k. JOINT AT J, b, & h THE SAME BUT WITH 3 COACH SCREWS.



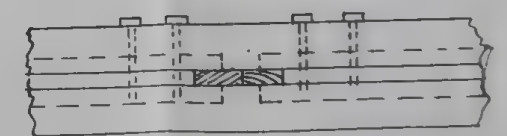
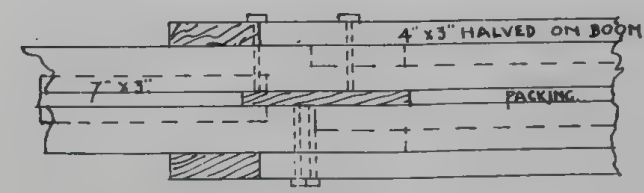
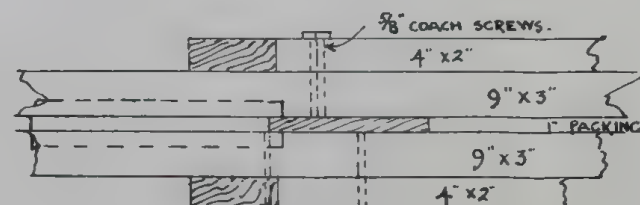
JOINT AT e.



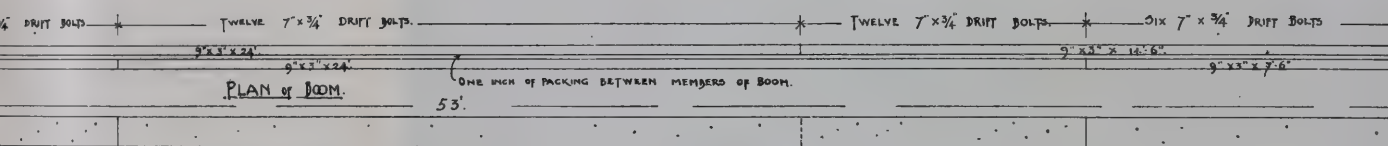
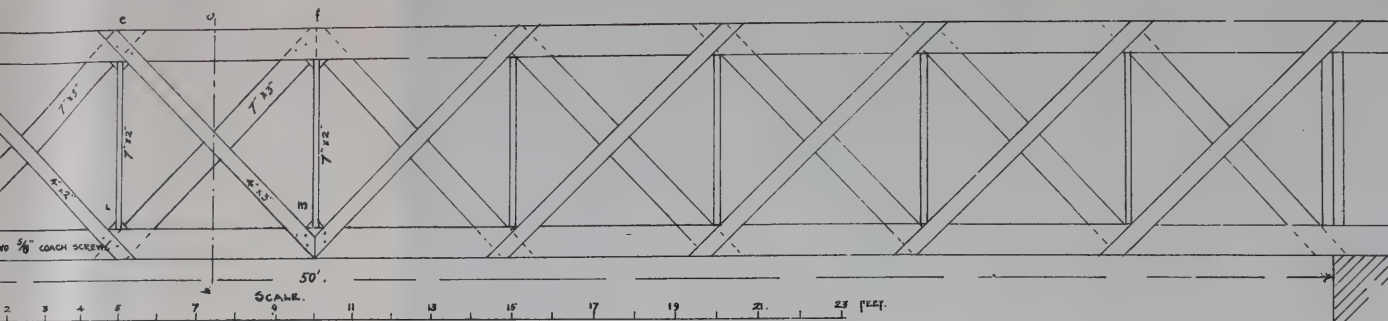
JOINT AT f.



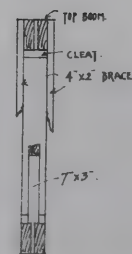
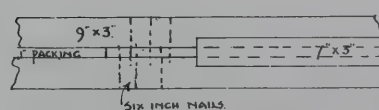
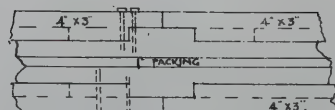
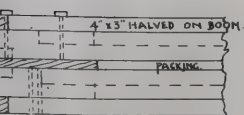
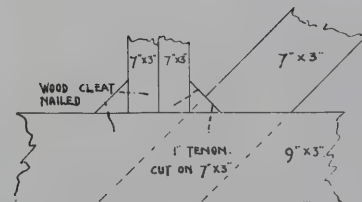
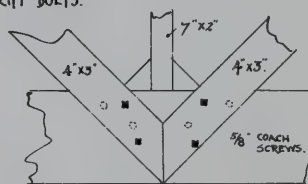
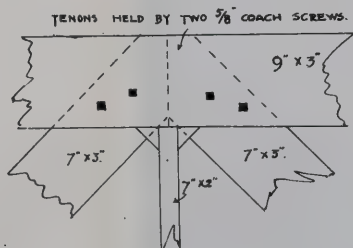
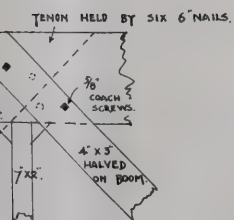
PLAN of JOINT at a.



CH GIRDERS TO CARRY TRAIN OF LORRIES, ROADWAY ON THE
TOP BOOM. STRESSES AS IN FIG. 13.



ELEVATION of BOOM, SHEWING ROUGHLY POSITION of DRIFT BOLTS.



SECTION on a-b

SECTION on b-c

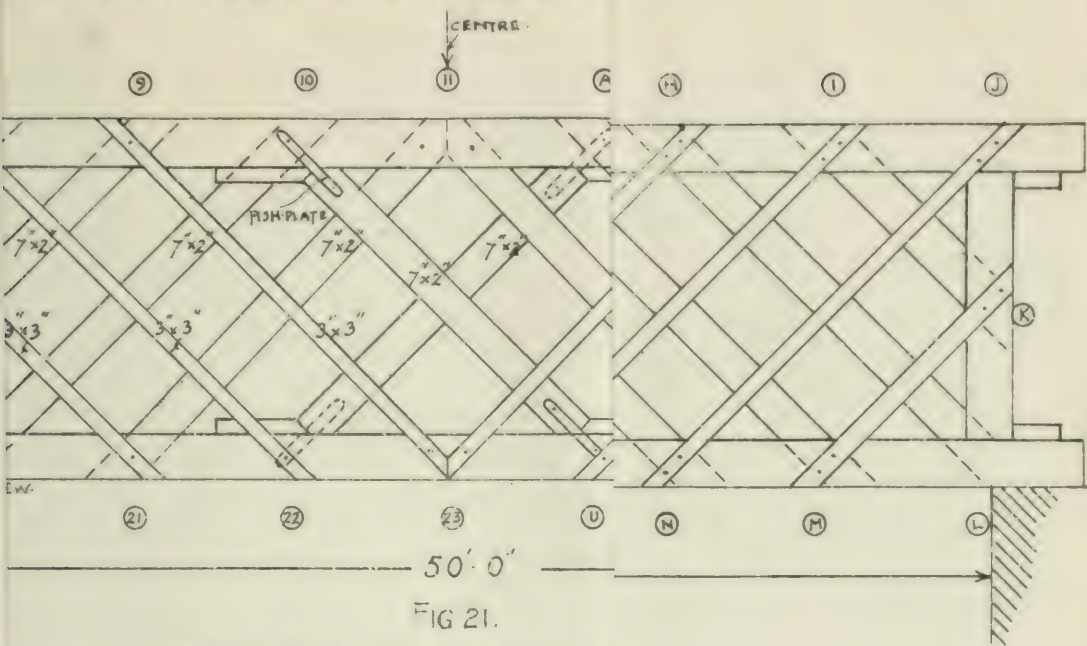
PLAN & ELEVATION of JOINT at g.

JOINT at f.

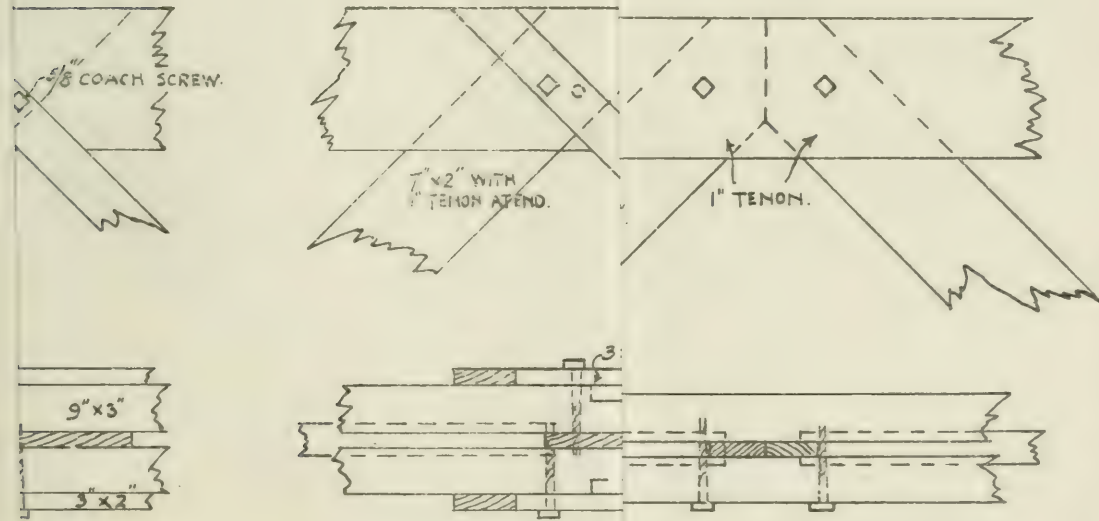
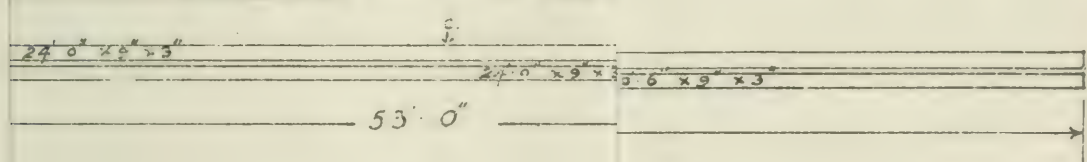
JOINT at m.

JOINT at e.

RS TO CARRY TRAIN OF HEAVIEST MOTOR



BETWEEN THESE JOINTS. 12. 3/4" ROUND I



5.17. H.N.O.P.
19.6.20. G.F.E.S.R.Q.
WITH ONLY ONE COACH SCREW.

JOINT AT 7.21. 8.9. JOINT AT 11.

PLATE VII.

1. CENTRE.

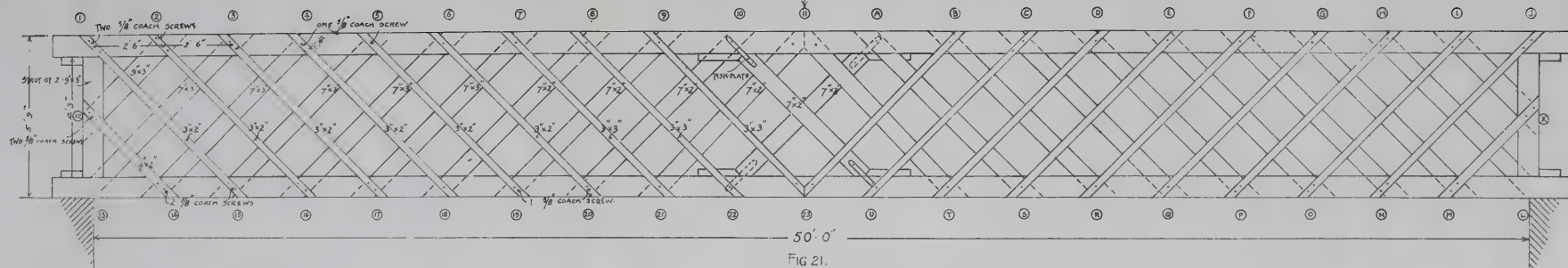
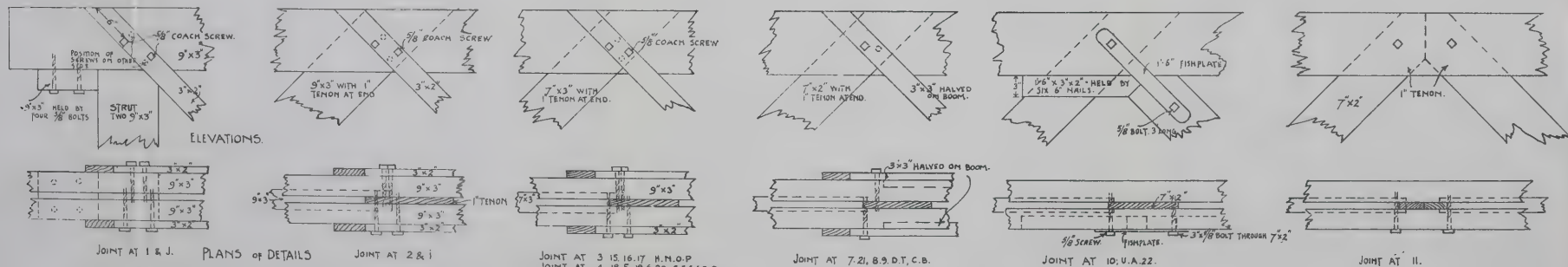
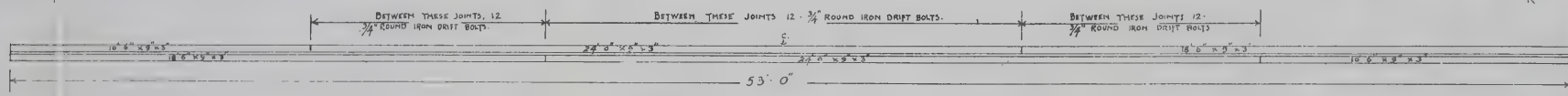


FIG 21.



50-FT. SPAN, TO TAKE HEAVY

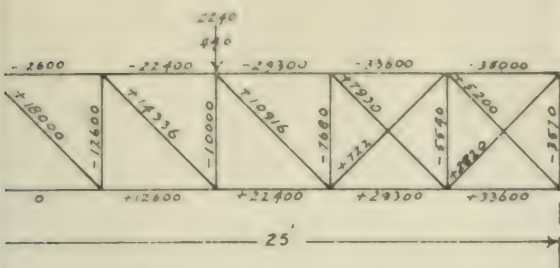
PLATE VIII.

Made of 55-lb. Rails and Telegraph

55 LB RAIL

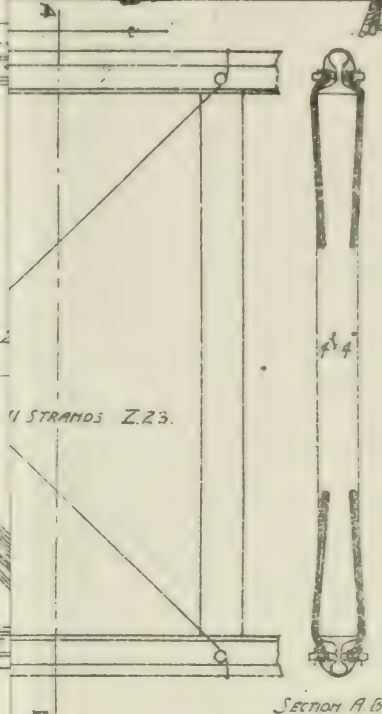
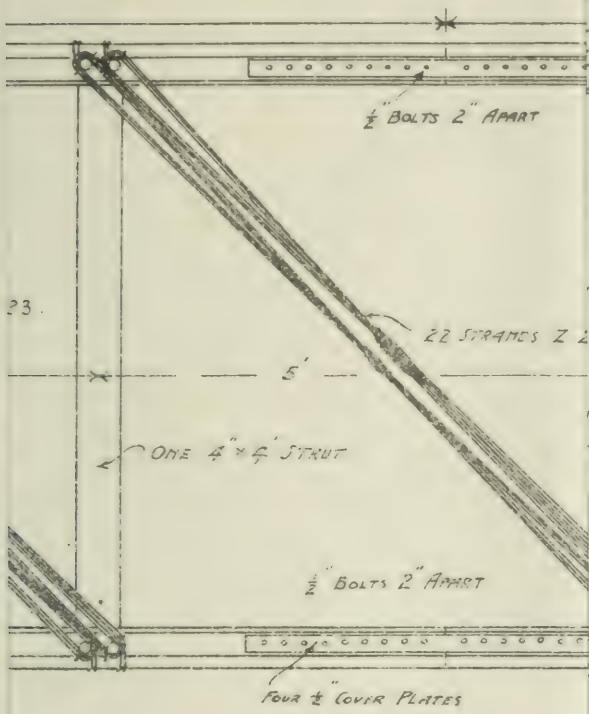
Z 23 WIRE

1/4" PEGS

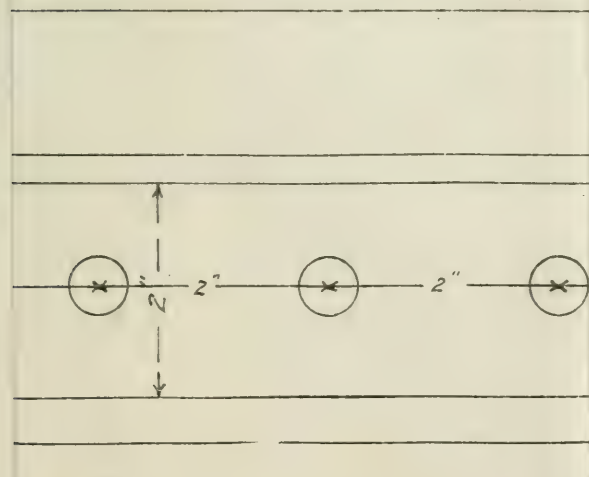


STRESS DIAGRAM FOR HALF SPAN.

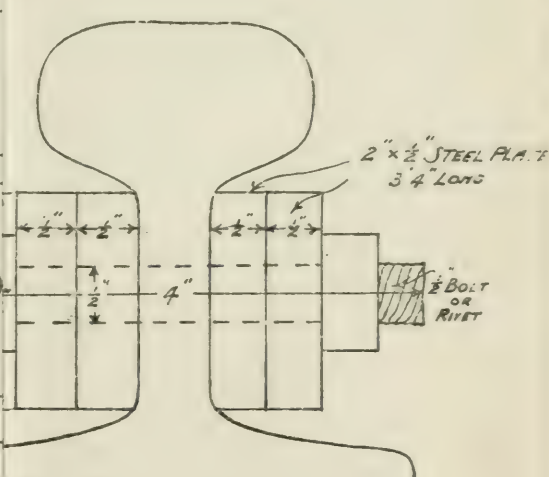
TAIL OF DIAGONAL JOINTS.



HALF ELEVATION OF TRUSS. SCHEMATIC



BOOM JOINT, FULL SIZE.



SECTION OF RAIL JOINT. FULL SIZE.

Made of 55-lb. Rails and Telegraph Wire.

55 LB RAIL

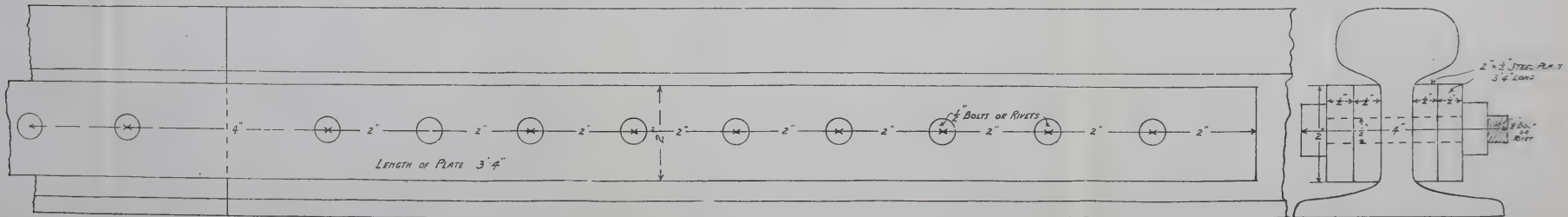
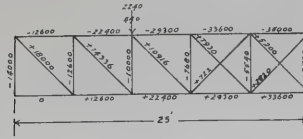
Z 23 WIRE

1/2" ROUND STEEL

5"

DETAIL OF DOUBLE JOINT.

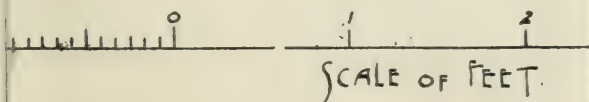
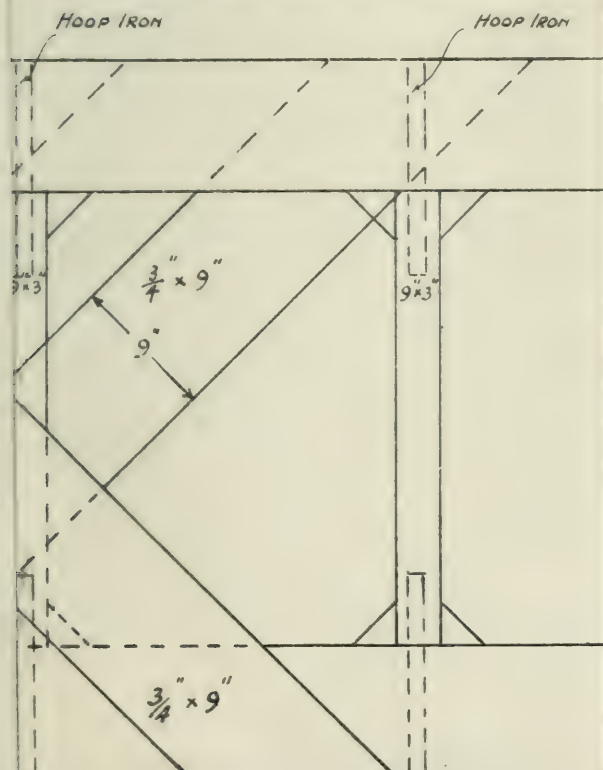
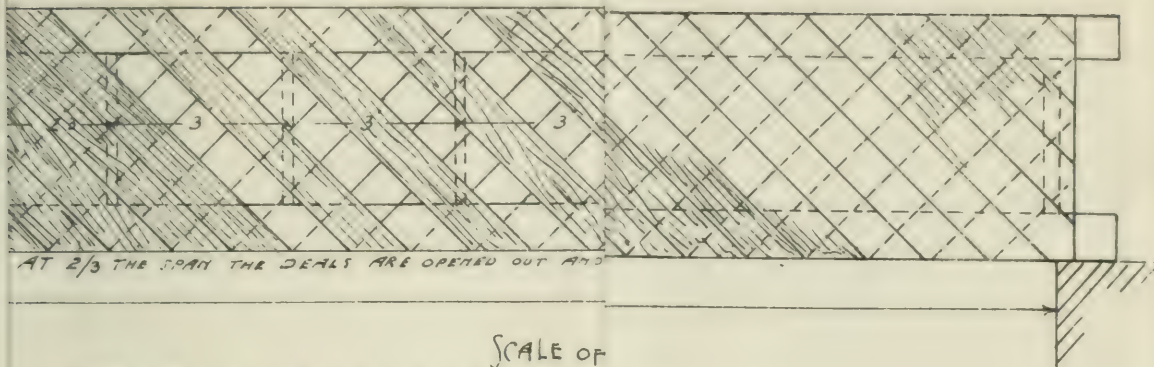
LENGTH OF RAIL 24"



SECTION OF RAIL JOINT.
FULL SIZE.

ELEVATION OF PLATE GIRDER. FOUR SUCH GIRDERS

BOOMS MADE UP OF THREE 9"



ELEVATION OF PLATE GIRDER. FOUR SUCH GIRDERS TO CARRY A TRAIN OF HEAVY LORRIES.

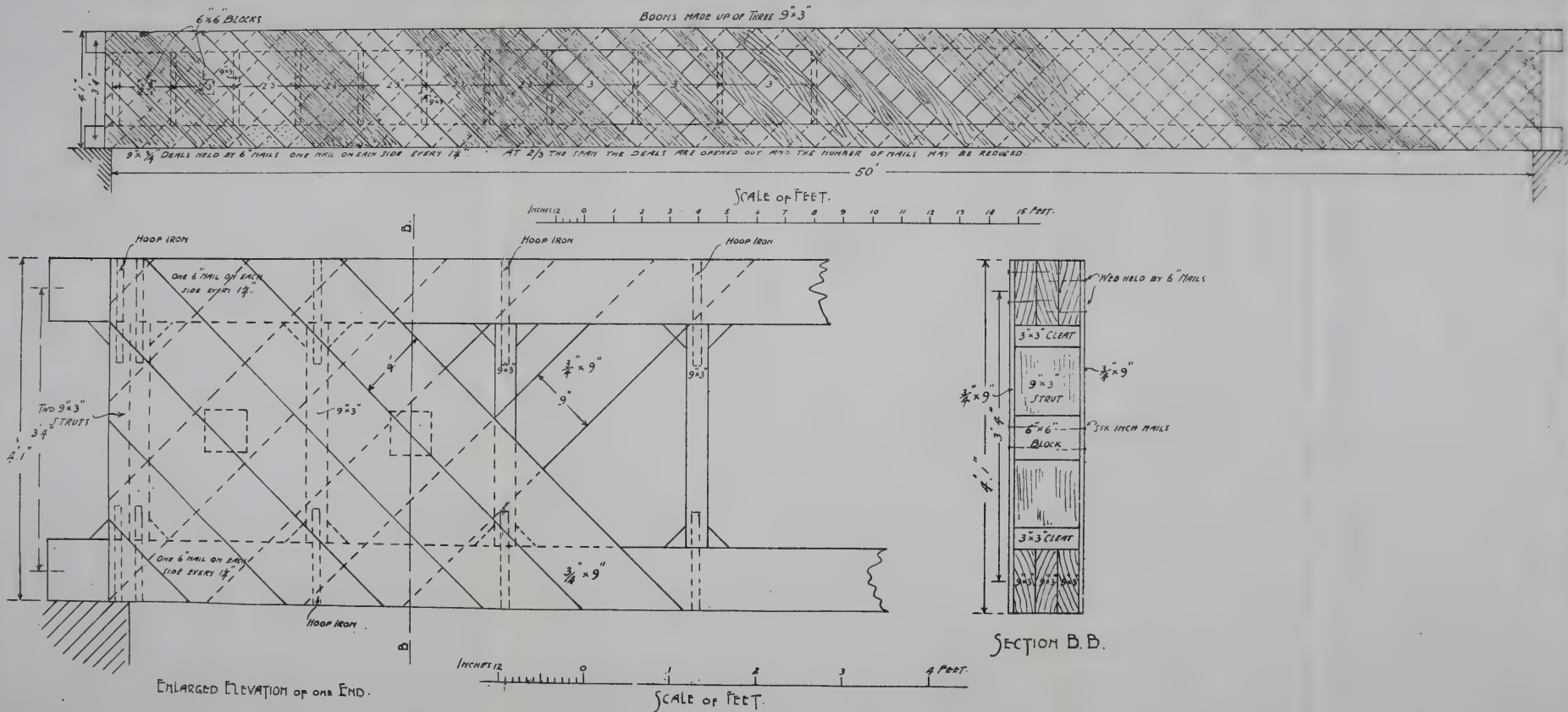


PLATE X.

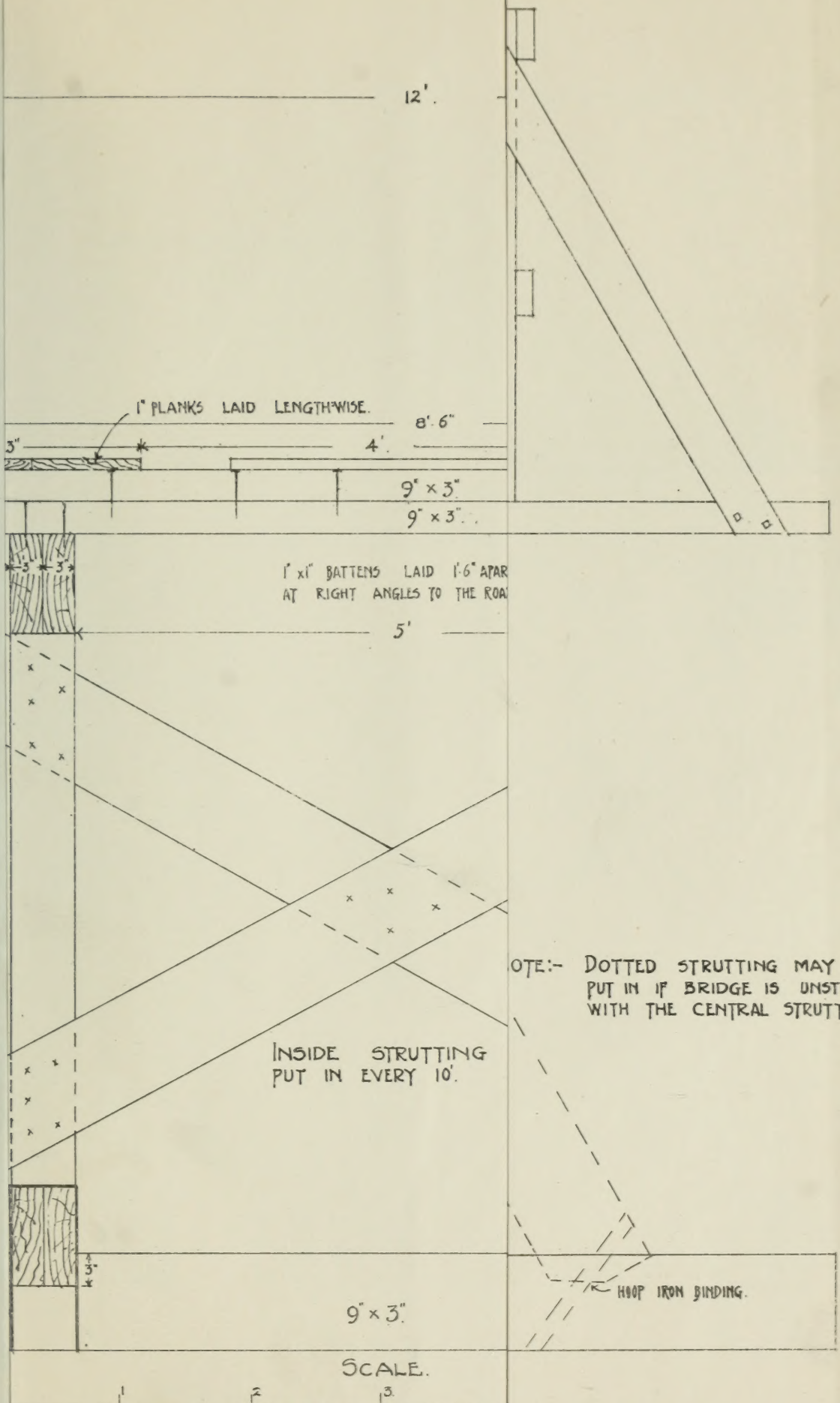


PLATE X.

